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REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

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CONTENTS

Papers: PAGE

Stresses in Helically Reinforced Concrete Columns.
By A. W. ZESIGER, M. AM. SOC. C. E., and E. J. AFFELDT, ASSOC. M. AM. SOC. C. E. 3

Evaporation on United States Reclamation Projects.
By IVAN E. HOUK, M. AM. SOC. C. E. 41

Pipe Tunnel Under Gowanus Canal, Brooklyn, New York.
By L. S. STILES, M. AM. SOC. C. E. 62

Discussions:

Fire-Banks for Oil Storage.
By MESSRS. RALPH J. REED and H. H. HALL 82

The St. Lawrence Waterway to the Sea.
By MESSRS. ELLIOTT J. DENT, KENNETH L. DE BLOIS, and C. S. RICHÉ 86

Multiple-Arch Dam at Gem Lake on Rush Creek in California.
By MESSRS. LUIGI LUIGGI, THADDEUS MERRIMAN, M. M. O'SHAUGHNESSY, J. B. LIPPINCOTT, and OREN REED 90

The Improved Venturi Flume.
By MESSRS. E. S. LINDLEY and C. E. CARTER 102

Side Channel Spillways.
By MESSRS. I. GUTMANN and A. J. WILEY 107

Permissible Canal Velocities.
By MESSRS. R. A. HILL, JEROME H. FERTIG, E. S. LINDLEY, and ANDREW WEISS 111

The Oradell Dam of the Hackensack Water Company.
By MESSRS. STUART K. KNOX, HOWARD T. CRITCHLOW, and A. T. RICKETTS. 118

Moments in Restrained and Continuous Beams by the Method of Conjugate Points.
By MESSRS. CHARLES S. WHITNEY, EDWARD GODFREY, FELIX H. SPITZER, and C. A. P. TURNER 122

Relation of the Ohio River and Its Tributaries to Transportation in the United States.
By MESSRS. M. G. BARNES and C. I. GRIMM 135

Co-Ordination of Irrigation and Power.
By MESSRS. J. D. GALLOWAY, FRED. H. TIBBETTS, BARRY DIBBLE, MARKHAM CHEEVER, and M. C. HINDERLIDER 145

The Hexagonal Slab Design of Concrete Pavement.
By E. G. HOOPER, ASSOC. M. AM. SOC. C. E. 158

Stream Pollution: A Symposium.
By HARRISON P. EDDY, M. AM. SOC. C. E. 162

The Effect of Moisture on Concrete.
By G. E. P. SMITH, M. AM. SOC. C. E. 164

Control of Odors from Sewage Treatment Plants.
By JOHN F. SKINNER, M. AM. SOC. C. E. 169

High Specific Speed Hydraulic Turbines: A Symposium.
By MESSRS. W. M. WHITE and LEWIS F. MOODY 171

Standard Specifications for Concrete and Reinforced Concrete; Report of Joint Committee on Standard Specifications for Concrete and Reinforced Concrete.
By L. T. GUY, ASSOC. M. AM. SOC. C. E. 174

Memoirs:

EDWARD ADAM BECK, M. AM. SOC. C. E. 175
ENRIQUE BUDGE, M. AM. SOC. C. E. 177
STEPHENSON WATERS FOX, M. AM. SOC. C. E. 181
CLARENCE LORING HUSSEY, M. AM. SOC. C. E. 183
VIRGIL GEORGE MARANI, M. AM. SOC. C. E. 185
MANSFIELD MERRIMAN, M. AM. SOC. C. E. 187
AMBROSE VINCENT POWELL, M. AM. SOC. C. E. 191
ROBERT HUGH MCNEILLY, ASSOC. M. AM. SOC. C. E. 192
ALBERT FRANKLIN SICKMAN, ASSOC. M. AM. SOC. C. E. 193

REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

This Society is not responsible for any statement made or opinion expressed in its publications.

CONTENTS

Papers: PAGE

Stresses in Helically Reinforced Concrete Columns.
By A. W. Zeasman, M. Am. Soc. C. E., and K. J. Arnvidt, Assoc. M. Am. Soc. C. E. 3

Evaporation on United States Irrigation Projects.
By Ivan E. Houk, M. Am. Soc. C. E. 41

Pipe Tunnel Under Gowanus Canal, Brooklyn, New York.
By L. S. Stuart, M. Am. Soc. C. E. 62

Discussions:

Fire-Hazards of Oil Storage.
By Messrs. Ralph J. Reed and H. H. Hird. 82

The St. Lawrence Waterway in the Sea.
By Messrs. Milnor J. Dyer, Kenneth I. De Bord, and C. S. Rich. 86

Multiple-Arch Dam at Gem Lake on Rush Creek in California.
By Messrs. Lloyd Lupton, Theodore Marshall, M. M. O'Shaughnessy, J. B. Lippincott, and Owen Hugh. 90

Side Channel Spillways.
By Messrs. I. Cotchaev and H. [illegible] 107

Permissible Canal Velocities.
By Messrs. R. A. Hart, Jerome D. Price, E. E. Lowder, and Andrew Weiss. 111

The Orchell Dam of the Hackensack Water Company.
By Messrs. Stuart K. Knox, Howard T. Critchlow, and A. T. Ricketts. 118

Moments in Restrained and Continuous Beams by the Method of Conjugate Points.
By Messrs. Charles S. Whitney, Edward Godfrey, Fritz H. Bertram, and C. A. P. Turner. 122

Relation of the Ohio River and Its Tributaries to Transportation in the United States.
By Messrs. M. O. Eaton and C. I. Quinn 136

Co-Ordination of Irrigation and Power.
By Messrs. J. D. Galloway, Fred. H. Tinkertt, Harry Dibble, Markham Chertkin, and M. C. Hinderlider. 145

The Hexagonal Slab Design of Concrete Pavement.
By E. C. Hooper, Assoc. M. Am. Soc. C. E. 158

Stream Pollution: A Symposium.
By Harrison P. Eddy, M. Am. Soc. C. E. 162

The Effect of Moisture on Concrete.
By G. E. P. Smith, M. Am. Soc. C. E. 164

Control of Odors from Sewage Treatment Plants.
By John P. Skinner, M. Am. Soc. C. E. 169

High Specific Speed Hydraulic Turbines: A Symposium.
By Messrs. W. M. White and Lewis F. Moody 171

Standard Specifications for Concrete and Reinforced Concrete: Report of Joint Committee on Standard Specifications for Concrete and Reinforced Concrete.
By L. T. Guy, Assoc. M. Am. Soc. C. E. 174

Memoirs:

Edward Adam Beck, M. Am. Soc. C. E. 175
Francis Hinde, M. Am. Soc. C. E. 177
Stephenson Wayne Fox, M. Am. Soc. C. E. 181
Clarence Loomis Hyatt, M. Am. Soc. C. E. 182
Virgil George Marani, M. Am. Soc. C. E. 186
Manfredo Merriman, M. Am. Soc. C. E. 187
Ambrose Vincent Powell, M. Am. Soc. C. E. 191
Robert Hugh McNeely, Assoc. M. Am. Soc. C. E. 192
Albert Franklin Bickman, Assoc. M. Am. Soc. C. E. 192

For Index to all Papers, the discussion of which is current in *Proceedings*, see the second page of the cover.

STRESSES IN HELICALLY REINFORCED CONCRETE COLUMNS*

By A. W. Zesiger,† M. Am. Soc. C. E., and E. J. Affeldt,‡ Assoc. M. Am. Soc. C. E.

Synopsis

The object of this paper is to analyze the effect of helical reinforcement on the concrete core of a helically reinforced concrete column, and to develop formulas for the stresses which obtain in the steel and concrete with varying percentages of reinforcement.

As the available tests on helically reinforced concrete columns are comparatively limited in number and have been made under such widely different conditions (as regards values of $\frac{l}{r}$, quality of materials, age of concrete, etc.) as to render their co-ordination extremely difficult, no claim is made for the absolute accuracy of the empirical constants hereinafter given. This paper is presented more with the idea of indicating the lines along which further research would be highly beneficial than as the final word on the subject. As a result of careful study of the available data, the writers believe, however, that the method of analysis proposed is substantially correct and that the following conclusions are justified:

(1) Plain concrete columns, of lengths up to at least 80 $\frac{l}{r}$, fail in shear rather than in compression.

(2) Before a plain concrete column can fail in shear, the imposed load must be sufficient to overcome the shearing resistance of the concrete and the frictional resistance of the material to incipient motion along the impending plane of rupture.

(3) The frictional resistance of the material is a more important factor than is commonly assumed.

(4) Helical reinforcement is effective in resisting a shearing failure of the concrete core and, if sufficient reinforcement is used, it is possible to preclude a shearing failure of the concrete.

Note.—Written discussion on this paper will be closed with the April, 1926, *Proceedings*. When finally closed, the paper, with discussion in full, will be published in *Transactions*.

* The word, helical, is used instead of the commonly employed term, spiral, because a spiral lies in one plane only and is, therefore, not the proper word to use in describing the system of reinforcement which will be discussed in this paper.

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(5) Helical reinforcement has little effect in preventing crushing of the concrete core of the column.

(6) Three types of failure may be observed in helically reinforced concrete columns, tested to destruction, the type of failure depending on the amount of helical reinforcement:

a.—When only a small amount of reinforcement is used (up to about 1%) failure of the column is due to the shearing resistance of the concrete and the frictional resistance of the material, to incipient motion along the impending plane of rupture, being exceeded.

b.—With larger percentages of reinforcement (from about 1% to about 1.6%) failure of the column is no longer due to shear, but to crushing of the concrete.

c.—With reinforcement in excess of about 1.6% the column will not fail, even after the concrete has crushed, until the reinforcement is stressed considerably beyond its elastic limit and possibly, in some cases, almost to its ultimate strength.

(7) Even if the concrete shell outside the reinforcement, in a helically reinforced column, cracks and spalls off under a load only little greater than is necessary to cause failure in an unreinforced column, of the same dimensions and quality of concrete, the reinforcement tends to neutralize the effects of local imperfections in the concrete, thus increasing the reliability of the column. Furthermore, unlike the plain concrete column which usually fails suddenly, warning of impending failure is given in the reinforced column by the cracking and spalling of the outer shell of concrete. It is permissible, therefore, to allow a lower factor of safety and correspondingly increased working stresses in helically reinforced columns.

(8) A rational analysis of the stresses in the steel and concrete of a helically reinforced concrete column is possible, contrary to the opinion of many engineers, and the resulting equations are very simple.

(9) There is need of further research work on helically reinforced concrete columns in order to evaluate more accurately the constants in the rational formulas.

Although this paper is primarily concerned with the analysis of helically reinforced concrete columns, an analysis of plain concrete columns is essential to an understanding of reinforced columns. It seems necessary, therefore, to consider, first, the unreinforced column with the object of ascertaining the relations existing therein, between stress and imposed load, and then to determine wherein these relations are modified by the introduction of reinforcement.

Authorities seem agreed that, for values of $\frac{l}{r}$ up to about 80, concrete columns may be assumed to act as short struts in which the effect of lateral flexure need not be considered. Furthermore, columns having a ratio of $\frac{l}{r}$ in

excess of 80 are seldom used in practice. It will be assumed, therefore, in the following discussion, that the columns are sufficiently short to preclude the necessity of considering lateral flexure.

I.—Plain Concrete Columns

Any theoretical analysis of concrete columns is of value only to the extent that it explains the phenomena observed in tests, and it is essential to the development of a rational theory that the manner in which the column actually fails under load be determined. Upon this point there is a reported disagreement in tests, and it has been commonly accepted that plain concrete columns may fail either in shear or by a gradual crushing of the material—the type of failure depending on the quality of the concrete.

In the writers' opinion, there is a greater disagreement in the reported results than in actual tests. The majority of available tests of plain concrete columns disclose an unmistakable shearing failure, and it is probable that, in a large proportion of the columns where failure has been ascribed to crushing of the concrete, a more careful diagnosis would have shown that the column failed in shear, a contention which is substantiated in many instances by photographs of the test specimens taken after failure. Moreover, a large proportion of the so-called crushing failures have occurred in columns composed of concretes too lean for actual construction work, and are not a fair criterion. It seems reasonable, therefore, to assume that the average unreinforced concrete column, constructed of a grade of concrete ordinarily used in good practice, will fail in shear; a crushing failure being an exception which is due to some pronounced defect in materials or workmanship.

The resistance which an unreinforced concrete column offers to a shearing failure consists of: (1) the shearing strength of the concrete; and (2) the frictional resistance of the material to incipient motion along the impending plane of rupture. If failure of the column was resisted solely by the shearing strength of the material the plane of rupture would make an angle of 45° with the horizontal, as the shearing component of the imposed load is a maximum along a plane having this inclination. Tests, however, show conclusively that the actual plane of rupture is inclined considerably more than 45° with the horizontal and the difference in inclination can only be ascribed to the frictional resistance of the material.

Until recently, writers, even Rankine, have accepted the erroneous theory of Coulomb wherein friction is neglected and the plane of rupture is assumed to make an angle of 45° with the horizontal, despite the fact that Navier has given the true analysis of granular substances in compression. Although engineers now admit the existence of friction in concrete, for some inexplicable reason they usually fail to give it proper consideration, as is evidenced by the frequent discussions regarding the cube *versus* the cylinder as the proper criterion of the compressive strength of a concrete column. The difference between the results obtained with cubical and cylindrical test specimens (having a height of 2 to 3 diameters) is fundamentally due to the effect of friction, a conclusion that may be inferred from a careful study of current literature, pertinent to the subject, which has not been clearly and concisely

stated. If the true significance of frictional resistance in the concrete was properly appreciated the fallacy of considering the cube as a criterion of the compressive strength of a column would be so self-evident as to be almost axiomatic and there would be no need of any discussion.

Tests show conclusively that the compressive strength developed by a cube exceeds that of a cylinder (having a height of 2 to 3 diameters), and that the results of cylinder tests agree reasonably close with those on actual columns of the same concrete, which is just what theory leads one to expect. Manifestly, the steepest plane of rupture, which can intersect the entire cross-section of a cube, has an inclination of 45° with the horizontal. Inasmuch as the plane of rupture concomitant with a shearing failure has a much steeper inclination, it is obviously impossible for a cube to fail in pure shear, and it is con-

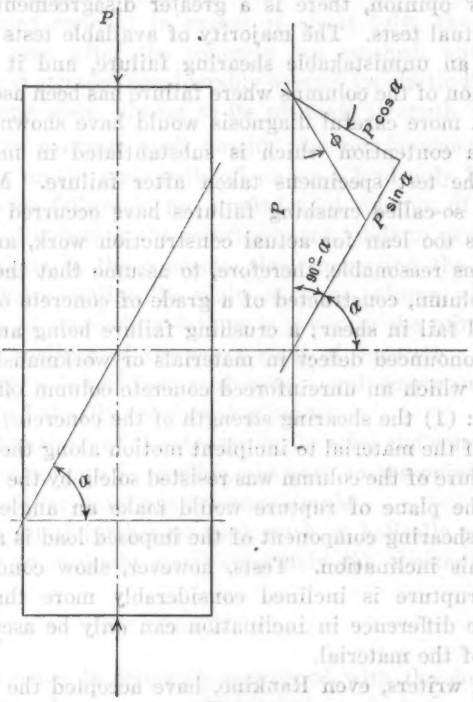

FIG. 1.

strained, therefore, to fail in combined shear and crushing. However, in a cylinder having a height of 2 to 3 diameters the entire cross-section can be intersected by a plane of rupture, having the inclination requisite to a true shearing failure, and the cylinder, therefore, will fail in shear alone. In other words, the difference in the mode of failure of cubes and cylinders and the discrepancy in compressive strength consequent thereon is due to the additional inclination of the plane of rupture, concomitant with a true shearing failure, which is introduced by the frictional resistance of the concrete.

Knowing the manner in which a plain concrete column fails, under imposed load, and the factors which tend to resist this failure, it becomes a simple

matter to deduce formulas for the stress in the concrete and the inclination of the plane of rupture.

Referring to Fig. 1, let,

P = the imposed load, in pounds.

A = the cross-sectional area of the column, in square inches.

$f_c' = \frac{P}{A}$ = the unit compressive stress in the concrete, in pounds per square inch.

f_z = the unit shearing stress in the concrete, in pounds per square inch.

α = the angle which the plane of rupture makes with a normal to the axis of the column.

ϕ = the friction* angle of the concrete.

The component of P, which tends to cause sliding along the plane of rupture, is $P \sin \alpha$. This tendency to slide is resisted by $f_z A \sec \alpha + P \cos \alpha \tan \phi$, in which, $\tan \phi$ equals the coefficient of friction. Hence, for equilibrium,

$$P \sin \alpha = f_z A \sec \alpha + P \cos \alpha \tan \phi \quad \ldots\ldots (1)$$

or,

$$f_z = \frac{P}{A} (\sin \alpha \cos \alpha - \cos^2 \alpha \tan \phi) \quad \ldots\ldots (2)$$

Rupture of the column will manifestly occur along that plane on which f_z is a maximum; consequently, at failure, $\frac{d f_z}{d \alpha} = 0$, whence,

$$\alpha = 45° + \frac{1}{2} \phi \quad \ldots\ldots (3)$$

From Equation (3) it is seen that the angle of rupture is 45° plus one-half the friction angle.

Substituting $\left(45° + \frac{1}{2} \phi\right)$ for α and $-\cot (90° + \phi)$ for $\tan \phi$ in Equation (2):

$$f_z = \frac{1}{2} \frac{P}{A} \cot \left(45° + \frac{1}{2} \phi\right) \quad \ldots\ldots (4)$$

and,

$$f_c' = \frac{P}{A} = 2 f_z \tan \left(45° + \frac{1}{2} \phi\right) \quad \ldots\ldots (5)$$

When the eccentricity of the imposed load is considerable, due either to the load not being applied along the axis of the column or to the non-homogeneity of the concrete, the maximum compressive stress on the extreme fiber may greatly exceed the unit compressive stress which, if uniformly distributed, would cause the column to shear across its entire section. Failure of the column would then be due to the progressive crushing or shearing of the extreme fibers. However, a shearing failure of the extreme fiber can not

* The term, friction angle, is used for lack of a better one. Strictly speaking, ϕ is an angle such that its tangent is equal to the coefficient of the friction in the concrete.

take place until a section in the shape of a wedge is sheared out, and this can not occur until the compressive stress on the extreme fiber becomes great enough to overcome the shearing strength of the concrete and the frictional resistance on the two faces of the wedge.

The angle of rupture, which is concomitant with the foregoing type of failure, may be determined as follows:

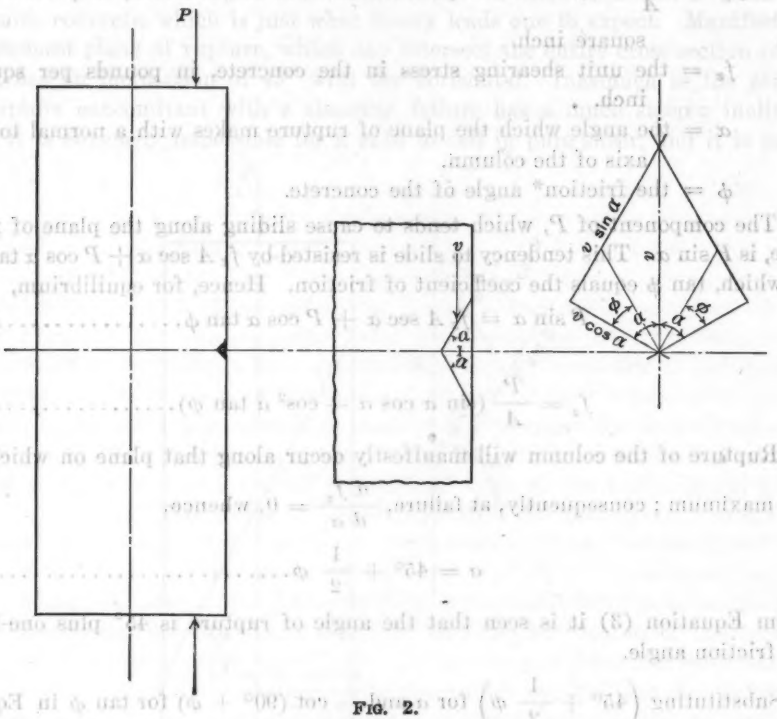

FIG. 2.

Referring to Fig. 2, let,

f_c = the maximum unit compressive stress on the extreme fiber, in pounds per square inch.

f_s = the unit shearing stress in the concrete, in pounds per square inch.

α = the angle which the two planes of rupture make with a normal to the axis of the column.

ϕ = the friction angle of the concrete.

The component of f_c, which tends to cause sliding along the two planes of rupture, is $f_c \sin \alpha$. This tendency to slide is resisted by $f_s \sec \alpha + 2 f_c \cos \alpha \tan \phi$. Hence, for equilibrium,

$$f_c \sin \alpha = f_s \sec \alpha + 2 f_c \cos \alpha \tan \phi \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

or,

$$f_s = f_c (\sin \alpha \cos \alpha - 2 \cos^2 \alpha \tan \phi) \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

As rupture will occur along the planes on which f_s is a maximum, it follows, at rupture, that $\frac{d f_s}{d \alpha} = 0$; whence,

$$\cot 2\alpha = -2 \tan \phi = 2 \cot (90° + \phi) \quad \ldots\ldots (8)$$

Substituting $\cot 2\alpha$ for $-2 \tan \phi$ in Equation (7),

$$f_s = \frac{1}{2} f_c \cot \alpha \quad \ldots\ldots (9)$$

or,

$$f_c = 2 f_s \tan \alpha \quad \ldots\ldots (10)$$

The relation between the unit compressive stress which will cause a shearing failure across the entire section of the column and that which will shear out a wedge in the extreme fiber may be determined, for any given value of ϕ, as follows: Dividing Equation (10) by Equation (5) and solving for f_c:

$$f_c = f_c' \frac{\tan \alpha}{\tan \left(45° + \frac{1}{2} \phi\right)}$$

If $\phi = 45°$ (a commonly used value for concrete), $\alpha = 76° 43'$ and $f_c = 1.75 f_c'$. In other words, the unit compressive stress which will cause failure by shearing out a wedge in the extreme fiber must be at least 1.75 times the unit compressive stress which will cause failure by shearing the entire section of the column.

Having considered the plain concrete column, the analysis of the helically reinforced column may now be undertaken in order to ascertain wherein the relations between stress and imposed load which obtain in the unreinforced column are modified by the lateral restraint which is introduced by helical reinforcement. Helically reinforced concrete columns without longitudinal steel will be considered first, and the effect of longitudinal reinforcement will be discussed subsequently.

II.—Helically Reinforced Concrete Columns

FIG. 3.

Tests indicate that helically reinforced concrete columns may fail in any one of three ways, the type of failure depending on the quantity of reinforcement. With steel up to about 1%, the column will fail in shear the same as a plain concrete column. With steel in excess of 1% and up to about 1.6%, the lateral restraint of the reinforcement is sufficient to preclude a shearing failure and the column fails through crushing of the concrete. With reinforcement in excess of about 1.6%, the column will not fail, even after the concrete has crushed, until the helical reinforcement has been stressed considerably beyond its elastic limit, causing undue elongation or rupture and thereby permitting the crushed concrete to slide.

The analysis of the helically reinforced concrete column, without longitudinal steel, is as follows:

Referring to Fig. 3, let,

P = the imposed load, in pounds.

H = the horizontal thrust in pounds, supplied by the helical reinforcement.

v = the unit vertical pressure in the concrete, due to P.

h = the unit horizontal pressure in the concrete, due to H.

v_t = the unit tangential component of v along a plane making an angle, α, with a plane normal to the axis of the column.

v_n = the unit normal component of v on the plane mentioned.

h_t = the unit tangential component of h along the plane mentioned.

h_n = the unit normal component of h on the plane mentioned.

f_z = the unit shearing stress in the concrete along the plane mentioned.

α = the angle which the plane of rupture (for a shearing failure) makes with a normal to the axis of the column.

ϕ = the friction angle of the concrete.

D = the diameter of the helix, in inches.

a = the cross-sectional area, in square inches, of the steel composing the helix, per unit of column length.

f_s = the unit stress in the helical reinforcement.

e_v = the unit vertical deformation, in inches, of the column.

e_h = the unit horizontal deformation, in inches, of the column.

m = Poisson's ratio.

E_s = the modulus of elasticity of steel.

E_c = the modulus of elasticity of concrete.

$n = \dfrac{E_s}{E_c}$

p' = the percentage of helical reinforcement.

The following equations are fundamental and, of course, require no explanation:

$$v_t = v \sin \alpha \cos \alpha$$
$$v_n = v \cos^2 \alpha$$
$$h_t = h \sin \alpha \cos \alpha$$
$$h_n = h \sin^2 \alpha$$

The tendency to slide on the plane of rupture is $v_t - h_t$. The resistance to sliding is $f_z + v_n \tan \phi + h_n \tan \phi$. Hence, for equilibrium,

$$v_t - h_t = f_z + (v_n + h_n) \tan \phi \quad \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

or,

$$f_z = v_t - h_t - (v_n + h_n) \tan \phi$$
$$= (v - h) \sin \alpha \cos \alpha - v \cos^2 \alpha \tan \phi - h \sin^2 \alpha \tan \phi \quad \ldots\ldots (12)$$

Failure of the column, through shearing, will occur along that plane on which f_z is a maximum; therefore, at failure, $\dfrac{d f_z}{d \alpha} = 0$; whence,

$$\alpha = 45^\circ + \frac{1}{2} \phi \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

From Equation (13) it is seen that, for a shearing failure, the inclination of the plane of rupture in a helically reinforced column, like that of a plain concrete column, is 45° plus one-half the friction angle.

In the analysis of the plain concrete column it was found, as shown by Equation (4), that the unit shearing stress is:

$$f_z = \frac{1}{2}\frac{P}{A} \times \cot\left(45° + \frac{1}{2}\phi\right)$$

The unit shearing stress in a helically reinforced column is:

$$f_z = (v - h)\sin\alpha\cos\alpha - v\cos^2\alpha\tan\phi - h\sin^2\alpha\tan\phi$$

as shown in Equation (12). For equal unit shearing stresses in the two types of columns these expressions become equal; whence,

$$\frac{1}{2}\frac{P}{A}\cot\left(45° + \frac{1}{2}\phi\right) = (v - h)\sin\alpha\cos\alpha - v\cos^2\alpha\tan\phi - h\sin^2\alpha\tan\phi \quad \ldots\ldots(14)$$

In this manner, the unit shearing stresses may be eliminated and a relation established between the concomitant unit compressive stresses. The unit compressive stress which will cause a shearing failure in a plain concrete column, may readily be determined from cylinder tests of the same grade of concrete and the concomitant value of f_z calculated by Equation (4). At failure, the shearing stress in the concrete becomes equal to its shearing strength, which is constant for any given concrete whether used in plain or reinforced columns. Consequently, having determined the unit compressive stress concomitant with shearing failure in a plain concrete column, it becomes a simple matter to determine the corresponding unit compressive stress in a helically reinforced column, by the relation between $\frac{P}{A}$ and v expressed in Equation (14). In a similar manner, the unit compressive stress, concomitant with any given shearing stress in the helically reinforced column, may be expressed in terms of the unit compressive stress, which will produce the same unit shearing stress in a plain concrete column; or, letting f_c' equal the unit compressive stress which will produce any given unit shearing stress in a plain concrete column:

$$\frac{1}{2}f_c'\cot\left(45° + \frac{1}{2}\phi\right) = (v - h)\sin\alpha\cos\alpha - v\cos^2\alpha\tan\phi - h\sin^2\alpha\tan\phi \quad \ldots\ldots(15)$$

Before Equation (15) can be solved for v, it is necessary to express h in terms of v. This may be accomplished by means of the relations which exist between the vertical and lateral deformations, and between the lateral deformation and the stress in the helix. It is shown in treatises on structural mechanics that,

$$e_v = \frac{v}{E_c} - \frac{2h}{mE_c} = \frac{1}{E_c}\left(v - \frac{2}{m}h\right) \quad \ldots\ldots(16)$$

and,

$$e_h = -\frac{h}{E_c} + \frac{h + v}{m\,E_c} = \frac{1}{E_c}\left(\frac{h + v}{m} - h\right) \quad \ldots\ldots (17)$$

In terms of the stress which it causes in the helix:

$$h = \frac{1}{D}\,(2\,\alpha\,e_h\,E_s) = \frac{1}{2}\,p'\,e_h\,E_s \quad \ldots\ldots (18)$$

or,

$$e_h = \frac{2\,h}{p'\,E_s} \quad \ldots\ldots (19)$$

Whence, from Equation (17):

$$h = \frac{v\,n\,p'}{2\,m + (m-1)\,n\,p'} \quad \ldots\ldots (20)$$

Substituting the value of h from Equation (20) and the value of α from Equation (13), in Equation (15), and solving for v:

$$v = \frac{f_c'}{1 - \dfrac{n\,p'\tan^2\left(45° + \dfrac{1}{2}\,\phi\right)}{2\,m + (m-1)\,n\,p'}} \quad \ldots\ldots (21)$$

When f_c' represents the unit vertical compressive stress required to produce a shearing failure in a plain concrete column, the corresponding stress, for a helically reinforced column, is represented by v in Equation (21). From Equation (21), it can readily be seen that v exceeds f_c' thus indicating the value of helical reinforcement in resisting a shearing failure. As the percentage of reinforcement increases, the denominator of the right-hand member of Equation (21) decreases, with a corresponding increase in the value of v. If the reinforcement is sufficient, v will become equal to the crushing strength of the concrete, in which case the column will no longer fail in shear, but by crushing of the concrete.

If v_c equals the compressive stress in plain concrete and e_v equals the corresponding deformation, Equation (16) becomes:

$$e_v\,E_c = v_c = v - \frac{2}{m}\,h = v\left\{1 - \frac{2\,n\,p'}{m\,[2\,m + (m-1)\,n\,p']}\right\} \quad \ldots\ldots (22)$$

Whence,

$$v = \frac{v_c}{1 - \dfrac{2\,n\,p'}{m\,[2\,m + (m-1)\,n\,p']}} \quad \ldots\ldots (23)$$

When the helical reinforcement is sufficient to preclude a shearing failure of the column, the vertical unit compressive stress, required to cause failure of the column by crushing, may be determined by Equation (23), when v_c represents the crushing strength of the same grade of plain concrete.

If the material within the helix is a granular mass, such as crushed concrete, the resistance to tangential movement or sliding is represented by Equation (12), in which f_z is now the resistance to sliding along the assumed

plane of rupture. As failure will occur along that plane on which f_s is a minimum, $\frac{df_s}{d\alpha} = 0$; whence,

$$\alpha = 45° + \frac{1}{2}\phi \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (24)$$

When the resistance to sliding is overcome, Equation (12) becomes equal to zero ; whence, if $\alpha = 45° + \frac{1}{2}\phi$:

$$v = h \tan^2 \left(45° + \frac{1}{2}\phi\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (25)$$

In terms of the stress in the helix, however:

$$h = \frac{1}{2} f_s p' \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (26)$$

Whence,

$$v = \frac{1}{2} f_s p' \tan^2 \left(45° + \frac{1}{2}\phi\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (27)$$

As long as the value of v, determined from Equation (23), is less than the value of v, determined from Equation (27), when f_s represents the stress in the steel as it reaches its yield point, failure of the column will not occur even if the concrete within the helix is crushed by the imposed load.

If, in Equation (21), $\tan^2 \alpha$ is substituted for $\tan^2 \left(45° + \frac{1}{2}\phi\right)$, where α is determined from Equation (8), v will be the compressive stress necessary to shear out a wedge under eccentric load.

When longitudinal steel is used in conjunction with helical reinforcement, Equation (21) becomes:

$$v = \frac{f_c' \left[1 + (n' - 1)\, p\right]}{1 - \dfrac{n\, p' \tan^2 \left(45° + \dfrac{1}{2}\phi\right)}{2\, m + (m - 1)\, n\, p'}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (28)$$

The theoretically correct equation is:

$$v = \frac{f_c' \left[1 + (n' - 1)\, p\right]}{1 - \dfrac{n\, p' \tan^2 \left(45° + \dfrac{1}{2}\phi\right)}{2\, m + (m - 1)\, n\, p'}} \times \left[1 - \frac{2\, n\, p'}{m \left[2\, m + (m - 1)\, n\, p'\right]}\right] \quad \ldots\ldots (29)$$

The difference, however, in the results obtained with Equation (29) and with Equation (28) is small, and Equation (28) is much simpler and sufficiently accurate for all practical purposes.

Also, when longitudinal steel is used in conjunction with helical reinforcement, Equation (23) becomes

$$v = \frac{v_c \left[1 + (n - 1)\, p\right]}{1 - \dfrac{2\, n\, p'}{m + \left[2\, m + (m - 1)\, n\, p'\right]}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (30)$$

in which, p equals the percentage of longitudinal steel.

Due to slipping, or some unknown cause, the longitudinal steel does not develop as much stress as is indicated by n at failure and, therefore, a lower value, n', must be used.

The theoretically correct equation is:

$$v = \frac{v_c}{1 - \frac{2\,n\,p'}{m\,[2\,m + (m - 1)\,n\,p']}} + v_c\,n\,p \quad \ldots\ldots\ldots\ldots (31)$$

but Equation (30) is used in order that the values of v, as determined from Equation (28) and Equation (30), respectively, may be equal for a certain value of p', regardless of the value of p. Tests (see graph of tests on helically reinforced concrete columns (Fig. 4) by Professor Wrentmore hereinafter discussed) indicate that curves, having as their equations, Equations (28) and (30), respectively, intersect when p' equals about 0.01. In other words, when p' equals about 0.01, a helically reinforced concrete column will not fail in shear before the concrete core has crushed.

From actual tests,* it appears that when $f_c' = 1\,490$, $n = 44$ and $m = 2.5$; and when $f_c' = 2\,833$, $n = 21$ and $m = 3$. In other words, n decreases as f_c' increases and m increases as f_c' increases, but there is not sufficient information available to give the relation between n and f_c', and m and f_c'. Until these relations are determined it seems advisable to use values for the aforesaid factors which obtain in the average concrete. Assuming as probable values, $n = 27$, $m = 2.5$, $n' = 20$, and $\phi = 50°$, substituting these values in Equations (28) and (30), respectively, and allowing a factor of safety of about 4†, the following working equations are obtained:

$$v = \frac{0.25\,f_c'\,(1 + 19\,p)}{1 - \frac{40\,p'}{1 + 8\,p'}} \quad \ldots\ldots\ldots\ldots (28a)$$

and,

$$v = \frac{0.383\,f_c'\,(1 + 19\,p)}{1 - \frac{4\,p'}{1 + 8\,p'}} \quad \ldots\ldots\ldots\ldots (30a)$$

A careful study of the test data, hereinafter given, indicates that n lies somewhere between 21 and 44, but nearer 21 than 44; and m lies somewhere between 2.5 and 3, but nearer 2.5. For the sake of convenience, m is assumed as 2.5, which is somewhat low, and to counteract this, the value of n is assumed as 27, which is undoubtedly less than the probable value. As previously stated, there appears to be a tendency for the longitudinal steel to slip, the amount of slip not remaining constant, and the value of n' is, therefore, indefinite. A study of tests, however, indicates that n' lies somewhere between 15 and 27. A value of 20 is chosen because, first, 20 is a convenient figure, and, second, its use in Equations (28a) and (30a) gives results which closely agree with the most modern practice. As regards the value of ϕ, tests indicate that, for

* See graph of tests (Fig. 4) conducted by Professor Wrentmore. These tests are reported in *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 97.

† The factor of safety is about 4 in Equation (28a), and about 4.5 in Equation (30a), a greater value being used in Equation (30a) because of the difficulty of determining the correct value of v_c.

percentages of reinforcement sufficient to preclude a shearing failure of the concrete, 50° is a reasonable value. For higher percentages of reinforcement, ϕ appears to be about 45 degrees. The only reason the writers can advance at present for the apparent decrease in the value of ϕ, for higher percentages of reinforcement, is that before the bond of the cement is broken the coarser particles of the aggregate dovetail into each other and are stronger in shear than the bonding cement. After the bond is broken, these particles have a tendency to roll upon each other, thereby diminishing the frictional resistance.

Although the values of n, m, n', and ϕ just given are only approximate, they are about as accurate as the available data will permit and, when substituted in Equations (28) and (30), the resulting values of v agree with actual tests as closely as may be expected when the great variation of these tests is taken into consideration. It should be clearly understood that the values of the aforesaid constants, which are used in obtaining the working Equations (28a) and (30a), are those that obtain at failure and the desired factor of safety is secured by using the proper coefficient of f_c'. Inasmuch as the values of n, m, and n' vary considerably as the stress in the concrete changes and are materially different at failure of the column than under working loads, the fallacy of attempting to secure a given factor of safety in a working formula by using therein the values of the aforesaid constants, which presumably obtain under working loads, should be self-evident. This point can not be stressed too strongly in view of the inexplicable tendency of many engineers to ignore it.

As helically reinforced concrete columns without longitudinal reinforcement are seldom if ever used, no attempt has been made to derive working equations for Equations (21) and (23).* As Equation (27) obtains only when the concrete core is crushed and is, therefore, only of academic interest and useless from the standpoint of design, a working equation of Equation (27) would be of no value.

A brief summary of various tests on concrete columns (principally helically reinforced concrete columns) is given in Table 1, wherein, for the sake of convenience in reference, the various columns are designated as Nos. (1) to (20), inclusive. Although the lengths of the test columns differed considerably and have not been noted in the table, it should be understood that no column had a ratio of $\frac{l}{r}$ exceeding the limits prescribed for short columns. For a few columns, the tests indicated that failure was due to flexure, but this was due, no doubt, to some local defect. It is understood that the limiting value of $\frac{l}{r}$ decreases as p' increases and theoretically becomes zero when p' is sufficiently large to preclude a shearing failure. Of course, even when $p' = 0.01$ (the limit of shearing failure), the column curve (for a crushing failure in the extreme fiber) is almost horizontal for a considerable value of $\frac{l}{r}$ and for

* Equations (28a) and (30a) become working formulas for Equations (21) and (23), respectively, when $p = 0$.

practical purposes the limiting value of $\frac{l}{r}$ would still be considerably greater than zero. As an aid to the understanding of the data presented in Table 1, the following explanatory notes may be of value:

The data given under Columns (1), (2), (3), (4), (5), (6), (7), (8), (9), (18), and (19), was furnished by the person, or persons, conducting the tests.

The ultimate strength of the column, as given under Column (10), was determined by dividing the maximum load by the area of the column cross-section. In other words, it would represent the ultimate strength of the column if the latter were perfectly homogeneous and each square inch of its cross-section were equally stressed.

The average cylinder test, given under Column (11), is the ultimate strength of the test specimen as determined by taking the average of all cylinder tests of a particular mixture of concrete in any given series of tests. It is, no doubt, unnecessary to state that, in all probability, concrete of given proportions in any one set of experiments will not be exactly the same as that of identical proportions in some other set of experiments.

The stress in the concrete, given under Column (12), was often rather difficult to determine, even approximately. Where longitudinal reinforcement was used, it was necessary first to determine (or assume) the stress in the longitudinal steel. Having determined (or assumed) this stress, it was then multiplied by the cross-sectional area of steel in order to obtain the total load carried by the longitudinal reinforcement. This load was then subtracted from the total load on the column and the remainder divided by the net cross-sectional area of the concrete in order to obtain the unit stress in the concrete.

The stress in the longitudinal steel, given under Column (13), was likewise rather difficult to determine. In certain series of tests (such as those by Withey), in which the amount of longitudinal steel varied while other conditions remained constant, the results were plotted on cross-section paper and a straight line passed through them in order to determine the increment of stress taken by the steel. In other cases, where the deformation was so great as to exceed considerably the elastic limit of the steel, some constant value a little in excess of the elastic limit, which best seemed to fit conditions, was assumed as the stress in the steel. In still other cases, where the elastic limit of the steel was not exceeded, the stress in the steel was determined by multiplying the observed unit deformation by the modulus of elasticity of the steel. In some instances, where the deformation was not given, the stress in the steel could not be determined.

The values of f_s, given under Column (14), were obtained from Equation (27) by using 45° as the value of ϕ and the results given under Column (12) as the values of v. The tests conducted by C. G. Wrentmore, M. Am. Soc. C. E., and noted in Table 1, indicate that 45° is the most probable value of ϕ in concrete that is already ruptured.

The values of $n f_c$, given under Column (15), were computed by the formula, $f = f_c [1 + (n - 1) p]$, in which, f is given under Column (10), f_c is given under Column (11), and n is the unknown quantity. When lateral reinforcement is

used, the formula just mentioned does not obtain and $n f_c$ can not be computed.

The values of n at failure, shown under Column (16), were obtained by exterpolation from the observed values of the tests whenever such data were given. No doubt these values are generally less than the real ones as the deformations become proportionately much greater at failure.

Having given the values of n, the values of m, given under Column (17), were then computed by Equation (21) wherein all the other quantities were assumed to be known. As may readily be seen, some values are less than those obtained by the assumption of constant volumes. However, the volume of concrete under compression always decreases; in such cases the computed values of m are, therefore, too small and either n, ϕ, or the strength of the concrete, are greater than the assumed values.

A study of Table 1 will readily indicate the difficulty of attempting to coordinate the majority of reported tests. Although there is considerable diversity in the mixture of concrete used and the age of the test specimens, there are only a few series of tests in which, with a constant mixture of concrete and age of test specimen, there is a sufficient variation in the percentage of helical reinforcement to indicate clearly what value of p' is the limit between Equation (21) and Equation (23), and what value of p' is the limit between Equation (23) and Equation (27). Inasmuch as v_c can only be determined when the exact intersection of curves, having as their equations, Equation (23) and Equation (27), respectively, is known, it can readily be seen how uncertain the value of v_c may be. The manner in which the helix influences the strength of the concrete may readily be seen, but the exact degree of influence is unknown as the exact values of m, n, ϕ, and v_c, to be used for the various concretes, are not known. Among the most instructive of the tests thus far reported are those which were conducted by Professor Wrentmore,* a graph of which is shown in Fig. 4.

In Fig. 4, the unit vertical pressures, sustained by the test columns at failure, were plotted as ordinates and the corresponding percentages of helical reinforcement as abscissas and curves, determined by Equations (21), (23), and (27), were then passed through the points thus obtained. For helical reinforcement up to about 1%, curves having as their equation, Equation (21), which is predicated upon a shearing failure of the column, seem to agree fairly well with the plotted tests. For reinforcement in excess of 1% and less than 2%, curves having as their equation, Equation (23), which is predicated upon a crushing failure of the concrete, seem to apply. For reinforcement in excess of from 1.6 to 2%, curves having as their equation, Equation (27), which is predicated upon the stress in the helical reinforcement, appear to be more nearly correct. On the basis of these tests, f_c', for 1 : 2 : 4 concrete, appears to lie somewhere between 1 490 and 2 833, n seems to lie somewhere between 21 and 44, m lies somewhere between 2.5 and 3, and ϕ is about 50° for the lower percentages of reinforcement and about 45° for percentages in

* See Table 1, also "Report of a Series of Tests on Concrete Columns Reinforced with a Spiral of Steel", by C. G. Wrentmore, M. Am. Soc. C. E., and Messrs. Hugh Brodie and C. O. Carey, *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 97.

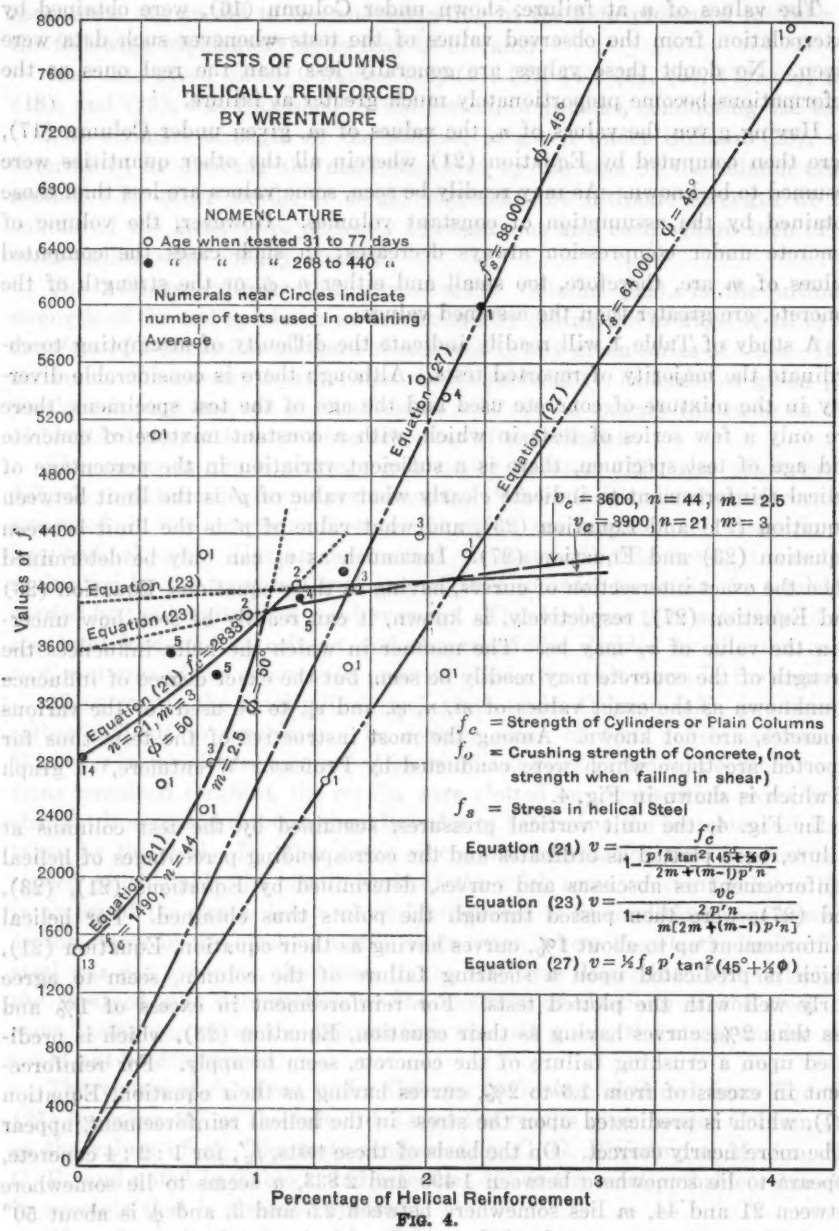
TESTS OF COLUMNS
HELICALLY REINFORCED
BY WRENTMORE
NOMENCLATURE
o Age when tested 31 to 77 days
● " " " 268 to 440 "
Numerals near Circles indicate number of tests used in obtaining Average
Values of f
Percentage of Helical Reinforcement
Equation (27)
Equation (23)
Equation (21) n=21, m=3
Equation (21) n=44, m=2.5
φ = 45°
φ = 50°
f_s = 88 000
f_s = 64 000
f_c = 2835
f_c = 1490
v_c = 3600, n = 44, m = 2.5
v_c = 3900, n = 21, m = 3
f_c = Strength of Cylinders or Plain Columns
f_o = Crushing strength of Concrete, (not strength when failing in shear)
f_s = Stress in helical Steel
Equation (21) v = f'_c / (1 − p'n tan²(45+½φ) / (2m+(m−1)p'n))
Equation (23) v = v_c / (1 − 2p'n / (m[2m+(m−1)p'n]))
Equation (27) v = ½ f_s p' tan²(45°+½φ)

FIG. 4.

excess of 1.6 per cent. The values of f_c', n, and m, seem to depend, to a great extent, on the age of the concrete. Regarding the apparent decrease in the value of ϕ for higher percentages of reinforcement, the only explanation which the writers can advance at present is that before the bond of the cement is broken the coarser particles of the aggregate dovetail into each other and are stronger in shear than the bonding cement. After this bond is broken, these particles have a tendency to roll upon each other, thereby diminishing the frictional resistance.

Although the available test data, pertaining to helically reinforced columns, is inadequate to evaluate accurately the various constants used in the foregoing equations, the writers feel that a careful study of the data, thus far available, will justify the following conclusions:

(1) Plain concrete columns, of lengths up to at least $80\ \frac{l}{r}$ tend to fail in shear, barring some pronounced defect in materials or workmanship.

(2) The resistance to failure offered by a plain concrete column consists of the shearing resistance of the concrete and the frictional resistance of the material to incipient motion along the impending plane of rupture.

(3) The frictional resistance of the concrete is an important factor in the strength of the column.

(4) Helical reinforcement is very effective in resisting a shearing failure of the concrete core of the column. If sufficient helical reinforcement is used, it is possible to preclude a shearing failure of the column.

(5) Helical reinforcement is not very effective in preventing crushing of the concrete core.

(6) Helical reinforcement tends to increase the reliability of a concrete column by neutralizing the effects of local imperfections. Furthermore, with helically reinforced columns, a warning of impending failure is given, at 70 to 90% of the ultimate strength, by the cracking or spalling of the outer shell. It is permissible, therefore, to allow a lower factor of safety and correspondingly increased working stresses in helically reinforced columns.

(7) Three types of failure occur in helically reinforced columns, the type of failure depending on the amount of reinforcement:

a.—With reinforcement up to about 1%, the column tends to fail in shear.

b.—With reinforcement from about 1% to about 1.6%, failure is due to the crushing of the concrete core of the column.

c.—With reinforcement in excess of about 1.6%, failure of the column will not occur even after the concrete is crushed, until the reinforcement is stressed considerably beyond its elastic limit and possibly, in some cases, up to its ultimate strength. Tests have been reported wherein the helix had ruptured under loads which, theoretically, could not stress the steel to anywhere near its ultimate strength. Such failures were undoubtedly due to a pronounced local defect in materials or workmanship which resulted in a local overstressing of the helix.

(8) The pitch of the helix materially affects the strength of the column, especially with higher percentages of reinforcement, and should be as small as practicable; the smaller the pitch of the helix the less will be the tendency for the particles of crushed concrete to flow between the coils of the helix.

(9) A rational analysis of the stresses in the steel and concrete of a helically reinforced concrete column is possible.

(10) Longitudinal reinforcement, consisting of rods or bars, should not be used unless it is well tied together at frequent intervals with lateral bands. It should preferably not be used except in conjunction with helical reinforcement.

(11) There is urgent need of additional research work on helically reinforced concrete columns, especially with reinforcement ranging from 0.75 to 2 per cent.

TABLE 1.—SUMMARY OF VARIOUS TESTS ON CONCRETE COLUMNS, PRINCIPALLY HELICALLY REINFORCED CONCRETE COLUMNS.

Source. (1)	Column designation. (2)	Reinforcement. Kind. (3)	Vertical percentage. (4)	Lateral percentage. (5)	Ratio of mix. (6)	Age. (7)	Area, in square inches. (8)	Maximum load, in pounds. (9)	Ultimate strength, in pounds per square inch. (10)	Average cylinder test. (11)	Stress in concrete. (12)	Stress in longitudinal steel. (13)	f_c (Equation (27)). (14)	$n f_c$.* (15)	n.† (16)	m. (17)	Cylinders. Age. (18)	Cylinders. Ultimate strength, in pounds per square inch. (19)	Remarks. (20)
Bulletin 300, University of Wisconsin, by Withey ⟶	A 1	None	0	0	1:2:4	65 days	144.0	294 000	2 040	.....	.....	.....	.....	.....	12.8	.....	63 days	2 427	.....
	A 2	"	0	0	1:2:4	63 "	143.0	303 000	2 110	.....	.....	.....	.....	.....	15.4	.....	63 "	2 395	.....
	A 3	"	0	0	1:2:4	64 "	142.0	292 000	2 055	.....	.....	.....	.....	.....	14.5	.....	62 "	2 240	.....
	A 4	"	0	0	1:2:4	65 "	141.0	294 000	2 080	.....	.....	.....	.....	.....	14.3	.....	62 "	2 360	.....
	Average		0	0	1:2:4	64¼ "	142.5	295 750	2 071	2 277†	.....	.....	.....	.....	14.1	.....	62½ "	2 363	.....
	B 1, B 4	Four, 2 by 2 in. by 5/16 in. latticed angles	4.5	0	1:2:4	64 "	64.0	237 000	3 700	.....	.....	.....	.....	.....	51.8	.....	62 "	2 270	.....
	B 4		4.5	0	1:2:4	63 "	64.0	242 000	3 780	.....	.....	.....	.....	.....	78.0	.....	62 "	2 140	.....
	Average		4.5	0	1:2:4	63½ "	64.0	239 500	3 740	2 277†	.....	.....	.....	34 900	64.9	.....	62 "	2 255	.....
	B 2, B 3	Four, 2 by 2 in. by 5/16 in. latticed angles	2.02	0	1:2:4	62 "	142.5	303 000	2 125	.....	.....	.....	.....	.....	.....	.....	62 "	2 640	Ultimate strength of column lower than cylinder tests.
	B 3		2.01	0	1:2:4	61 "	143.0	288 000	2 015	.....	.....	.....	.....	.....	.....	.....	61 "	2 205	
	Average		2.015	0	1:2:4	61½ "	142.75	.....	2 065	2 277†	.....	.....	.....	.....	.....	.....	61½ "	2 423	.....
	C 1	¼ in. high carbon steel, 10 in. in diameter; 1 in. pitch	0	2.00	1:2:4	62 "	78.54	366 000	4 660	.....	.....	.....	.....	.....	.....	.....	63 "	2 280	.....
	C 2		0	2.00	1:2:4	62 "	78.54	345 000	4 390	.....	.....	.....	.....	.....	.....	.....	62 "	2 190	.....
	C 3		0	2.00	1:2:4	64 "	78.54	288 000	3 660	.....	.....	.....	.....	.....	.....	.....	63 "	2 185	.....
	C 4		0	2.00	1:2:4	65 "	78.54	268 000	3 410	.....	.....	.....	.....	.....	.....	.....	62 "	2 150	.....
	Average		0	2.00	1:2:4	63¼ "	78.54	.....	4 030	2 277†	4 030	.....	39 100	.....	.....	.....	62½ "	2 201	.....
	D 1, D 2, D 3, D 4	Nine, 5/8 in. round mild steel rods and ¼ in. high-carbon steel spiral, 10 in. in diameter; 1 in. pitch	2.35	2.00	1:2:4	58 "	78.54	351 000	4 470	.....	.....	.....	.....	.....	.....	.....	56 "	2 150	.....
	D 2		2.35	2.00	1:2:4	60 "	78.54	330 000	4 200	.....	.....	.....	.....	.....	.....	.....	56 "	2 130	.....
	D 3		2.35	2.00	1:2:4	61 "	78.54	391 000	4 970	.....	.....	.....	.....	.....	.....	.....	56 "	2 380	.....
	D 4		2.35	2.00	1:2:4	62 "	78.54	421 000	5 360	.....	.....	.....	.....	.....	.....	.....	56 "	2 335	.....
	Average		2.35	2.00	1:2:4	60¼ "	78.54	373 250	4 750	2 277†	4 030	20 130	69 300	.....	.....	.....	56 "	2 259	Equation (27)

TABLE 1.—(*Continued.*)

Source.	Column designation.	Reinforcement. Kind.	Vertical percentage.	Lateral percentage.	Ratio of mix.	Age.	Area, in square inches.	Maximum load, in pounds.	Ultimate strength, in pounds per square inch.	Average cylinder test.	Stress in concrete.	Stress in longitudinal steel.	f_s (Equation (27)).	$n\,f_c$.*	n.†	m.	Cylinders. Age.	Cylinders. Ultimate strength, in pounds per square inch.	Remarks.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
	E 1	Nine, 5/8 in. round mild steel rods, with 1/4 in. ties 1 ft c. to c.	2.35	0.165	1 : 2 : 4	55 days	78.54	273 000	3 475								55 days	2 480	
	E 2		2.35	0.165	1 : 2 : 4	57 "	78.54	273 000	3 475								55 "	2 480	
	E 3		2.35	0.165	1 : 2 : 4	57 "	78.54	318 000	4 050								54 "	2 040	
		Average	2.35	0.165	1 : 2 : 4	56 1/3 "	78.54	288 000	3 667	2 277†							54 2/3 "	2 300	
	H 1	No. 7 wire spiral, 2 in. pitch	0	0.50	1 : 2 : 3.5	57 "	78.54	183 000	2 330						37		57 "	2 038	
	H 2		0	0.50	1 : 2 : 3.5	57 "	78.54	168 000	2 140						39		57 "	1 460	
		Average	0	0.50	1 : 2 : 3.5	57 "	78.54	175 500	2 235	1 881	2 235				38	3.535	57 "	1 749	
	G 1	Eight, 1/2 in. rods and No. 7 wire spiral, 2 in. pitch	2.00	0.50	1 : 2 : 3.5	49 "	78.54	261 000	3 320						35		49 "	2 095	
	G 2		2.00	0.50	1 : 2 : 3.5	58 "	78.54	258 000	3 280						37		57 "	1 430	
		Average	2.00	0.50	1 : 2 : 3.5	53 1/2 "	78.54	259 500	3 300	1 881	2 374	49 000			36	3.092	53 "	1 763	
	I 1	Eight, 1 1/16 in. round rods and No. 7 wire spiral, 2 in. pitch	3.78	0.50	1 : 2 : 3.5	57 "	78.54	330 000	4 240						36		59 "	2 235	
	I 2		3.78	0.50	1 : 2 : 3.5	57 "	78.54	321 000	4 080						32		59 "	2 115	
		Average	3.78	0.50	1 : 2 : 3.5	57 "	78.54	325 500	4 160	1 881	2 378	49 000			34	2.916		2 175	
	J 1	Eight, 7/8 in. round rods and No. 7 wire spiral, 2 in. pitch	6.11	0.50	1 : 2 : 3.5	58 "	78.54	408 000	5 190						27		59 1/2 "	2 210	
	J 2		6.11	0.50	1 : 2 : 3.5	58 "	78.54	397 500	5 050						54		59 "	1 995	
		Average	6.11	0.50	1 : 2 : 3.5	58 "	78.54	402 750	5 120	1 881	2 273	49 000			40.5	4.180	59 1/4 "	2 102	
	L 1	No. 7 wire spiral, 1 in. pitch	0	1.00	1 : 2 : 3.5	57 "	78.54	219 000	2 680						65		58 "	1 780	
	L 2		0	1.00	1 : 2 : 3.5	58 "	78.54	204 000	2 600						65		58 "	1 770	
		Average	0	1.00	1 : 2 : 3.5	57 1/2 "	78.54	211 500	2 640	1 881	2 640				65		58 "	1 775	Equation (23) or Equation (27).

TABLE 1.—(*Continued.*)

Source.	Column designation.	Reinforcement. Kind.	Reinforcement. Vertical percentage.	Reinforcement. Lateral percentage.	Ratio of mix.	Age.	Area, in square inches.	Maximum load, in pounds.	Ultimate strength, in pounds per square inch.	Average cylinder test.	Stress in concrete.	Stress in longitudinal steel.	f_s (Equation (27)).	$n f_c$.*	n.	m.	Cylinders. Age.	Cylinders. Ultimate strength, in pounds per square inch.	Remarks.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
Bulletin 166, University of Wisconsin, by Withey	*K* 1	Eight, ½ in. rods and No. 7 wire spiral, 1 in. pitch	2.00	1.00	1:2:3.5	57 days	78.54	318 000	4 050	.....	.....	.....	.....	.....	68	.....	58 days	2 100	.....
	K 2		2.00	1.00	1:2:3.5	57 "	78.54	295 500	3 760	.....	.....	.....	.....	.....	68	.....	58 "	1 895	.....
		Average	2.00	1.00	1:2:3.5	57 "	78.54	306 750	3 905	1 881	3 517	23 100	.....	.....	68	4.20	58 "	1 998	Equation (23)
	N 1	Eight, 11/16 in. rods and No. 7 wire spiral, 1 in. pitch	3.78	1.00	1:2:3.5	57 "	78.54	318 000	4 050	.....	.....	.....	.....	.....	46	.....	57 "	1 885	.....
	N 2		3.78	1.00	1:2:3.5	58 "	78.54	340 500	4 340	.....	.....	.....	.....	.....	63	.....	57 "	1 720	.....
		Average	3.78	1.00	1:2:3.5	57½ "	78.54	329 250	4 195	1 881	3 452	23 100	.....	.....	54	3.59	57 "	1 802	Equation (23)
	M 1	Eight, ⅞ in. rods and No 7 wire spiral, 1 in. pitch	6.11	1.00	1:2:3.5	57 "	78.54	376 500	4 790	.....	.....	.....	.....	.....	65	.....	57 "	1 665	.....
	M 2		6.11	1.00	1:2:3.5	58 "	78.54	360 000	4 580	.....	.....	.....	.....	.....	67	.....	57 "	1 710	.....
		Average	6.11	1.00	1:2:3.5	57½ "	78.54	368 250	4 685	1 881	3 493	23 100	.....	.....	66	4.16	57 "	1 688	Equation (23)
	P 1	Eight, 1 in. rods and No. 7 wire spiral, 1 in. pitch	8.00	1.00	1:2:4	57 "	78.54	531 300	6 760	.....	.....	.....	.....	.....	44	.....	57 "	2 385	.....
	P 2		8.00	1.00	1:2:4	60 "	78.54	556 500	7 090	.....	.....	.....	.....	.....	35	.....	57 "	2 360	.....
		Average	8.00	1.00	1:2:4	58½ "	78.54	542 750	6 925	2 380	3 500	46 270	.....	.....	39.5	4.10	57 "	2 372	Equation (23)
	O 1	Eight, ⅞ in. rods and ¼ in. spiral, 1 in. pitch	6.11	1.96	1:2:4	57 "	78.54	511 500	6 510	.....	.....	.....	.....	.....	.....	.....	57 "	2 270	.....
	O 2		6.11	1.96	1:2:4	57 "	78.54	522 000	6 650	.....	.....	.....	.....	.....	.....	.....	57 "	2 690	.....
		Average	6.11	1.96	1:2:4	57 "	78.54	516 750	6 580	2 380	5 278	26 600	92 600	.....	.....	4.16	57 "	2 480	Equation (23)
	R 1	Eight, 1 in. rods and ¼ in. spiral, 1 in. pitch	8.00	1.96	1:2:4	53 "	78.54	570 000	7 250	.....	.....	.....	.....	.....	.....	.....	53 "	2 320	.....
	R 2		8.00	1.96	1:2:4	53 "	78.54	525 000	6 680	.....	.....	.....	.....	.....	.....	.....	54½ "	2 475	.....
		Average	8.00	1.96	1:2:4	53 "	78.54	547 500	6 965	2 380	5 288	26 600	92 200	.....	.....	.....	53¾ "	2 398	Equation (27)
	Q 1	Eight, 1⅛ in. rods and ¼ in. spiral, 1 in. pitch	10.12	1.96	1:2:4	53 "	78.54	486 000	6 190	.....	.....	.....	.....	.....	.....	.....	53 "	2 275	.....
	Q 2		10.12	1.96	1:2:4	53 "	78.54	627 000	7 990	.....	.....	.....	.....	.....	.....	.....	54½ "	2 325	.....
		Average	10.12	1.96	1:2:4	53 "	78.54	556 500	7 090	2 380	4 894	26 600	85 700	.....	.....	.....	53¾ "	2 300	Equation (27)

TABLE 1.—(*Continued.*)

Source.	Column designation.	Reinforcement. Kind.	Vertical percentage.	Lateral percentage.	Ratio of mix.	Age.	Area, in square inches.	Maximum load, in pounds.	Ultimate strength, in pounds per square inch.	Average cylinder test.	Stress in concrete.	Stress in longitudinal steel.	f_s (Equation (27)).	nf_c.*	n.†	m.	Cylinders. Age.	Cylinders. Ultimate strength, in pounds per square inch.	Remarks.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
	W 1	None	0	0	1 : 2 : 4	52 days	86 6	231 000	2 660	.....	.....	.....	.....	.....	10.35	.....	49 days	2 605	.....
	W 2	"	0	0	1 : 2 : 4	52 "	86.6	231 000	2 660	.....	.....	.....	.....	.....	9.81	.....	49 "	2 395	.....
	W 3	"	0	0	1 : 2 : 4	51 "	86.6	214 500	2 480	.....	.....	.....	.....	.....	9.36	.....	49 "	2 050	.....
		Average	0	0	1 : 2 : 4	51½ "	86.6	.....	2 600	2 380	.....	.....	.....	.....	9.84	.....	49 "	2 350	.....
	Z 1	Eight, ⅞ in. rods and No. 7 high carbon steel wire, 1 in. pitch.	5.83	1.00	1 : 3 : 6	48 "	82.6	476 000	5 760	.....	.....	.....	.....	.....	63.6	.....	45 "	2 020	.....
	Z 2		5.83	1.00	1 : 3 : 6	49 "	82.6	411 000	4 980	.....	.....	.....	.....	.....	62.7	.....	45 "	1 520	.....
		Average	5.83	1.00	1 : 3 : 6	48½ "	82.6	443 510	5 370	1 770	4 920	50 000 (Assumed)	.....	.....	63.1	2.22	45 "	1 770	.....
	X 1	No. 7 high carbon steel wire, 1 in. pitch.	0	1.00	1 : 1¾ : 3¼	56 "	82.6	523 000	6 330	.....	.....	.....	.....	.....	11.45	.....	55 "	4 730	.....
	X 2		0	0.99	1 : 1¾ : 3¼	57 "	84.6	500 000	5 900	.....	.....	.....	.....	.....	10.32	.....	55 "	4 665	.....
		Average	0	1.00	1 : 1¾ : 3¼	56½ "	83.6	.....	6 165	4 433	6 165	.....	.....	.....	10.9	1.46	55 "	4 698	.....
	Y 1	Eight, ⅞ in. rods and No. 7 high carbon steel wire, 1 in. pitch.	5.69	0.99	1 : 1¾ : 3¼	60 "	84.6	617 500	7 300	.....	.....	.....	.....	.....	20.0	.....	62½ "	4 220	.....
	Y 2		5.83	1.00	1 : 1¾ : 3¼	62 "	82.6	617 500	7 480	.....	.....	.....	.....	.....	19.5	.....	62½ "	4 115	.....
		Average	5.76	1.00	1 : 1¾ : 3¼	61 "	83.6	617 500	7 390	4 433	6 165	27 300	.....	.....	19.75	2.50	62½ "	4 168	.....
	S 1	No. 7 high carbon steel wire, 1 in. pitch.	0	1.00	1 : 1 : 2	57 "	78.54	468 000	5 950	.....	.....	.....	.....	.....	14.15	.....	61 "	4 065	.....
	S 2		0	1.00	1 : 1 : 2	57 "	78.54	453 000	5 760	.....	.....	.....	.....	.....	7.05	.....	61 "	4 080	.....
		Average	0	1.00	1 : 1 : 2	57 "	78.54	.....	5 855	4 288	5 855	.....	.....	.....	10.6	1.47	61 "	4 072	.....
	T 1	Eight ⅞ in. rods and No. 7 high carbon steel wire, 1 in. pitch.	6.11	1.00	1 : 1 : 2	57 "	78.54	588 000	7 480	.....	.....	.....	.....	.....	8.86	.....	65 "	4 715	.....
	T 2		6.11	1.00	1 : 1 : 2	56 "	78.54	558 000	7 100	.....	.....	.....	.....	.....	7.20	.....	65 "	4 080	.....
		Average	6.11	1.00	1 : 1 : 2	56½ "	78.54	573 000	7 240	4 288	5 855	29 500	.....	.....	8.03	1.046	65 "	4 503	.....
	V 1	No. 7 high carbon steel wire.....	0	1.00	1 : 1.29	56 "	78.54	458 000	5 760	.....	.....	.....	.....	.....	8.31	.....	55 "	4 830	.....
	V 2		0	1.00	1 : 1.29	57 "	78.54	387 000	4 920	.....	.....	.....	.....	.....	7.37	.....	55 "	4 925	.....
		Average	0	1.00	1 : 1.29	56½ "	78.54	.....	5 340	4 708	5 340	.....	.....	.....	7.84	2.43	55 "	4 865	.....

TABLE 1.—(*Continued.*)

Source.	Column designation.	Reinforcement. Kind.	Vertical percentage.	Lateral percentage.	Ratio of mix.	Age.	Area, in square inches.	Maximum load, in pounds.	Ultimate strength, in pounds per square inch.	Average cylinder test.	Stress in concrete.	Stress in longitudinal steel.	f_o (Equation (27)).	nf_o*.	nl.	m.	Cylinders. Age.	Cylinders. Ultimate strength, in pounds per square inch.	Remarks.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
	U 1	Eight, ⅞ in. rods and No. 7 high carbon steel wire, 1 in. pitch.	6.11	1.00	1 : 1.29	56 days	78.54	633 000	8 050	.....	.....	.....	.....	.....	.....	.....	55 days.	4 900	.....
	U 2		6.11	1.00	1 : 1.29	57 "	78.54	648 000	8 250	.....	.....	.....	.....	.....	.....	.....	55 "	4 205	.....
		Average.....	6.11	1.00	1 : 1.29	56½ "	78.54	640 500	8 150	4 708	5 430	51 500	.....	.....	8.46	2.61	55 "	4 552	.....
	AA 1	Eight, ½ in. rods and No. 7 high carbon steel wire, 1 in. pitch.	1.86	0.99	1 : 1.33	57 "	84.6	617 500	7 300	.....	.....	35 000	.....	.....	4.8	.....	59 "	6 135	Column did not fail.
	AA 2		1.86	0.99	1 : 1.33	57 "	84.6	617 500	7 300	.....	.....	35 800	.....	.....	4.9	.....	59 "	7 430	
		Average.....	1.86	0.99	1 : 1.33	57 "	84.6	617 500	7 300	6 783	6 783	35 400	.....	.....	4.85	.....	59 "	6 783	.....
	AC 1	Eight, ⅞ in. rods and No. 7 high carbon steel wire, 1 in. pitch.	5.83	1.00	1 : 1⅛	53 "	82.6	617 000	7 480	.....	.....	35 200	.....	.....	4.71	.....	52 "	6 075	Column did not fail.
	AC 2		5.56	0.97	1 : 1⅛	54 "	86.6	617 500	7 140	.....	.....	35 440	.....	.....	4.97	.....	52 "	5 835	
		Average.....	5.70	0.985	1 : 1⅛	53½ "	84.6	617 250	7 310	5 955	.....	35 320	.....	.....	4.84	.....	52 "	5 955	.....
	111	None.....	0	0	1 : 2 : 4	66 "	113.2	240 000	2 120	.....	.....	.....	.....	.....	20.52	.....	.....	.....	.....
	112	"	0	0	1 : 2 : 4	62 "	113.2	281 000	2 480	.....	.....	.....	.....	.....	13.10	.....	.....	.....	.....
		Average.....	0	0	1 : 2 : 4	64 "	113.2	.....	2 300	2 300	.....	.....	.....	.....	16.8	.....	.....	.....	.....
	101	None.....	0	0	1 : 2 : 4	58 "	113.2	132 000	1 165	.....	.....	.....	.....	.....	21.15	.....	.....	.....	.....
	102	"	0	0	1 : 2 : 4	69 "	109	218 000	2 000	.....	.....	.....	.....	.....	16.38	.....	.....	.....	.....
	103	"	0	0	1 : 2 : 4	65 "	113.2	250 000	2 210	.....	.....	.....	.....	.....	16.01	.....	.....	.....	.....
	104	"	0	0	1 : 2 : 4	64 "	113.2	180 000	1 590	.....	.....	.....	.....	.....	16.05	.....	.....	.....	.....
	105	"	0	0	1 : 2 : 4	62 "	113.2	220 000	1 945	.....	.....	.....	.....	.....	20.86	.....	.....	.....	.....
	108	"	0	0	1 : 2 : 4	72 "	113.2	165 000	1 460	.....	.....	.....	.....	.....	13.9	.....	.....	.....	.....
	109	"	0	0	1 : 2 : 4	64 "	113.2	205 000	1 810	.....	.....	.....	.....	.....	17.6	.....	.....	.....	.....
		Average.....	0	0	1 : 2 : 4	65 "	113.2	.....	1 740	1 740	.....	.....	.....	.....	17.4	.....	.....	.....	.....

TABLE 1.—(*Continued.*)

Source. (1)	Column designation. (2)	Reinforcement. Kind. (3)	Reinforcement. Vertical percentage. (4)	Reinforcement. Lateral percentage. (5)	Ratio of mix. (6)	Age. (7)	Area, in square inches. (8)	Maximum load, in pounds. (9)	Ultimate strength, in pounds per square inch. (10)	Average cylinder test. (11)	Stress in concrete. (12)	Stress in longitudinal steel. (13)	f_s (Equation (27)). (14)	nf_c*. (15)	n ‖. (16)	m. (17)	Cylinders. Age. (18)	Cylinders. Ultimate strength, in pounds per square inch. (19)	Remarks. (20)
Bulletin No. 20, University of Illinois	116	None	0	0	1:3:6	61 days	113.2	108 000	955						17.62				
	117	"	0	0	1:3:6	62 "	113.2	126 000	1 110						18.80				
		Average	0	0	1:3:6	61½ "	113.2	.....	1 032	1 032					18.21				
	121	None	0	0	1:4:8	63 "	113.2	65 000	575						30.25				
	122	"	0	0	1:4:8	63 "	113.2	65 000	575						88.85				
		Average	0	0	1:4:8	63 "	113.2	.....	575	575					39.55				
	110	None	0	0	1:2:4	203 "	113.2	218 000	1 925						18.07				
	128	"	0	0	1:2:4	194 "	113.2	209 000	1 845						18.85				
	129	"	0	0	1:2:4	181 "	113.2	200 000	1 770						15.57				
	163	"	0	0	1:2:4	187 "	113.2	308 000	2 680						18.40				
	164	"	0	0	1:2:4	187 "	113.2	245 000	2 160						16.02				
	166	"	0	0	1:2:4	201 "	113.2	200 000	1 770						22.62				
		Average	0	0	1:2:4	192 "	113.2	.....	2 025	2 025					18.4				
	21	None	0	0	1:2:3½	12 mos.	144	381 500	2 650						20.25				
	22	"	0	0	1:2:3½	16 "	140.4	389 000	2 770						17.05				
		Average	0	0	1:2:3½	14 "	142.3	.....	2 710	2 710					18.65				
	131	No. 16 bands (mild steel) 2 in. c. to c.	1.08	1.08	1:2:4	60 days	113.2	270 000	2 384						4.67				
	132		1.08	1.08	1:2:4	57 "	113.2	243 000	2 150						6.75				
	133		1.05	1.05	1:2:4	59 "	113.2	247 000	2 182						6.91				
		Average	0	1.07	1:2:4	58½ "	113.2	253 333	2 239	2 239	1 740	2 876			6.18	1.117			
	136	No. 12 bands (mild steel) 2 in. c. to c.	2.08	2.08	1:2:4	63 "	113.2	323 000	2 860						6.73				
	137		2.07	2.07	1:2:4	69 "	113.2	300 000	2 660										
	138		2.12	2.12	1:2:4	60 "	113.2	351 000	3 110										
		Average	0	2.09	1:2:4	64 "	113.2	.....	2 876	2 876	1 740	2 876	.....	47 300	.....	10.86	2.05	.....	Equation (27)

TABLE 1.—(*Continued.*)

Source.	Column designation.	Reinforcement. Kind.	Vertical percentage.	Lateral percentage.	Ratio of mix.	Age.	Area, in square inches.	Maximum load, in pounds.	Ultimate strength, in pounds per square inch.	Average cylinder test.	Stress in concrete.	Stress in longitudinal steel.	f_s (Equation (27)).	$n f_c$.	n.	m.	Cylinders. Age.	Cylinders. Ultimate strength, in pounds per square inch.	Remarks.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
	146	No. 8 bands (mild steel), 2 in. c. to c.	0	3.22	1 : 2 : 4	60 days	113.2	339 000	3 000	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	147		0	3.20	1 : 2 : 4	66 "	113.2	420 000	3 715	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	148		0	3.20	1 : 2 : 4	63 "	113.2	327 000	2 890	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
		Average	0	3.207	1 : 2 : 4	63 "	113.2	.....	3 202	1 740	3 202	.....	34 250	.....	9.61	2.345	.....	.....	Equation (27).
	141	No. 12 bands (mild steel), 4 in. c. to c.	0	1.02	1 : 2 : 4	59 "	113.2	256 000	2 275	.....	.....	.....	.....	.....	7.90	.....	.....	.....	.....
	142		0	1.02	1 : 2 : 4	66 "	113.2	246 000	2 178	.....	.....	.....	.....	.....	7.99	.....	.....	.....	.....
		Average	0	1.02	1 : 2 : 4	62½ "	113.2	251 000	2 227	1 740	2 227	.....	.....	.....	7.95	1.887	.....	.....	.....
	148	No. 12 bands (mild steel), 3 in. c. to c.	0	1.39	1 : 2 : 4	60 "	113.2	309 000	2 735	1 740	2 735	.....	.....	.....	8.48	1.207	.....	.....	.....
	152	No. 16 bands (mild steel), 2 in. c. to c.	0	1.35	1 : 4 : 8	70 "	.....	86 000	1 345	.....	.....	.....	.....	.....	21.12	.....	.....	.....	.....
	153		0	1.41	1 : 4 : 8	56 "	.....	80 000	1 260	.....	.....	.....	.....	.....	23.93	.....	.....	.....	.....
		Average of 152 and 153	0	1.38	1 : 4 : 8	63 "	.....	83 000	1 303	575	1 303	.....	32 400	.....	22.5	1.95	.....	.....	Equation (27).
	151	No. 16 bands (mild steel), 2 in. c. to c.	0	1.47	1 : 2 : 4	65 "	.....	187 000	2 140	1 740	2 140	.....	.....	.....	5.69	1.66	.....	.....	Equation (23) or Equation (27).
	156	No. 12 bands (mild steel), 2 in. c to c.	0	2.76	1 : 2 : 4	67 "	.....	190 000	2 970	1 740	2 970	.....	37 300	.....	6.98	1.665	.....	.....	Equation (27).
	157		0	2.94	1 : 1½ : 3	67 "	.....	228 000	3 561	.....	.....	.....	.....	.....	6.51	.....	.....	.....	.....
	158		0	2.80	1 : 1½ : 3	81 "	.....	234 000	3 685	.....	.....	.....	.....	.....	5.37	.....	.....	.....	.....
		Average of 157 and 158	0	2.87	1 : 1½ : 3	74 "	.....	231 000	3 623	2 300	3 623	.....	43 300	.....	5.94	1.708	.....	.....	Equation (27).
	171	No. 7 wire spiral (high carbon steel)	0	0.85	1 : 2 : 4	56 "	.....	283 000	2 503	.....	.....	.....	.....	.....	7.76	.....	.....	.....	.....
	172		0	0.85	1 : 2 : 4	63 "	.....	283 000	2 506	.....	.....	.....	.....	.....	9.84	.....	.....	.....	.....
		Average	0	0.85	1 : 2 : 4	59½ "	.....	.....	2 505	1 740	2 505	.....	.....	.....	8.80	0.926	.....	.....	.....

TABLE 1.—(*Continued.*)

Source.	Column designation.	Reinforcement. Kind.	Vertical percentage.	Lateral percentage.	Ratio of mix.	Age.	Area, in square inches.	Maximum load, in pounds.	Ultimate strength, in pounds per square inch.	Average cylinder test.	Stress in concrete.	Stress in longitudinal steel.	f_s (Equation (27)).	$n f_s$.	n.	m.	Cylinders. Age.	Cylinders. Ultimate strength, in pounds per square inch.	Remarks.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
	176	No. 7 wire spiral (high carbon steel)	0	0.84	1:2:4	60 days	.....	235 000	2 080	.....	.....	.....	.....	.....	11.15	.....	.....	.....	.....
	177		0	0.85	1:2:4	64 "	.....	249 000	2 203	.....	.....	.....	.....	.....	7.90	.....	.....	.....	.....
	178		0	0.84	1:2:4	61 "	.....	251 000	2 220	.....	.....	.....	.....	.....	10.85	.....	.....	.....	.....
		Average	0	0.843	1:2:4	61⅔ "	.....	.....	2 168	1 740	2 168	.....	.....	.....	9.97	1.583	.....	.....	.....
	181	¼ in. spiral, 1 in. pitch (high carbon steel)	0	1.73	1:2:4	57 "	.....	307 000	2 718	.....	.....	.....	.....	.....	13.66	.....	.....	.....	.....
	182		0	1.67	1:2:4	60 "	.....	429 000	3 800	.....	.....	.....	.....	.....	9.16	.....	.....	.....	.....
	183		0	1.68	1:2:4	63 "	.....	428 000	3 793	.....	.....	.....	.....	.....	11.62	.....	.....	.....	.....
		Average	0	1.69	1:2:4	60 "	.....	.....	3 437	1 740	3 437	.....	69 800	.....	11.48	1.44	.....	.....	Equation (23) or Equation (27).
	186	¼ in. spiral, 1 in. pitch (mild steel)	0	1.64	1:2:4	58 "	.....	234 000	2 068	.....	.....	.....	.....	.....	16.12	.....	.....	.....	.....
	187		0	1.71	1:2:4	62 "	.....	385 000	3 401	.....	.....	.....	.....	.....	8.69	.....	.....	.....	.....
		Average	0	1.675	1:2:4	60 "	.....	.....	2 736	1 740	2 736	.....	56 000	.....	12.40	2.05	.....	.....	Equation (23) or Equation (27).
	1 682	None	0	0	1:2:0	6 mos. 0 days	159.38	395 000	2 500	3 358	.....	.....	.....	.....	8.34	4.69§	.....	.....	.....
	1 680	"	0	0	1:3:0	5 mos. 28 days	156.37	322 500	2 062	2 177	.....	.....	.....	.....	6.76	9.00§	.....	.....	.....
	1 679	"	0	0	1:4:0	5 mos. 21 days	156.0	244 000	1 564	1 408	.....	.....	.....	.....	22.55	8.21§	.....	.....	.....
	1 681	"	0	0	1:3:0	5 mos. 27 days	157.13	423 000	2 692	2 177	.....	.....	.....	.....	5.40	8.85§	.....	.....	.....
	1 685	"	0	0	1:4:0	6 mos. 9 days	157.75	232 100	1 471	1 408	.....	.....	.....	.....	12.82	5.79§	.....	.....	.....
	1 686	"	0	0	1:5:0	6 mos. 9 days	157.0	168 000	1 038	1 072	.....	.....	.....	.....	11.50	6.28§	.....	.....	.....

TABLE 1.—(*Continued.*)

TABLE 1.—(*Continued.*)

Source.	Column designation.	Reinforcement. Kind.	Vertical percentage.	Lateral percentage.	Ratio of mix.	Age.	Area, in square inches.	Maximum load, in pounds.	Ultimate strength, in pounds per square inch.	Average cylinder test.	Stress in concrete.	Stress in longitudinal steel.	f_s (Equation (27)).	$n f_c$.*	n.†	m.	Cylinders. Age.	Cylinders. Ultimate strength, in pounds per square inch.	Remarks.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
and Other Materials, Watertown Arsenal, 1905	1 673	Eight, ¾ in. square twisted bars	2.845	0	1:2:0	5 mos. 10 days	158.13	664 146	4 200	3 358	.....	60 500	.....	31 870	28.7	.....	.....	.....	.....
	1 690		2.845	0	1:3:0	7 mos. 21 days	157.0	608 000	3 841	2 177	.....	61 400	.....	60 650	28.5	8.79§	.....	.....	.....
	1 691		2.83	0	1:4:0	7 mos. 25 days	157.25	581 100	3 377	1 408	.....	53 800	.....	71 150	28.16	7.11§	.....	.....	.....
	1 688		2.826	0	1:5:0	6 mos. 17 days	157.50	443 000	2 813	1 072	.....	56 100	.....	68 100	40.45	.....	.....	.....	.....
	1 689	Thirteen, ¾ in. square twisted bars	4.63	0	1:5:0	7 mos. 15 days	157.75	616 000	3 905	1 072	.....	58 200	.....	62 400	45.90	.....	.....	.....	.....
	1 656	None	0	0	1:1:2	5 mos. 7 days	159.14	620 646	3 900	3 580	.....	.....	.....	.....	10.6	.....	.....	.....	.....
	1 683	"	0	0	1:2:4	6 mos. 5 days	161.38	321 200	1 990	1 827	.....	.....	.....	.....	12.8	.....	.....	.....	.....
	1 650	"	0	0	1:1:2	5 mos. 7 days	160.02	244 000	1 525	3 580	.....	.....	.....	.....	10.8	.....	.....	.....	.....
	1 651	"	0	0	1:2:4	5 mos. 6 days	160.64	242 000	1 506	1 827	.....	.....	.....	.....	19.2	.....	.....	.....	.....
	1 667	"	0	0	1:2:5	3 mos. 8 days	133.6	146 960	1 100	1 100	.....	.....	.....	.....	15.7	.....	.....	.....	.....
	1 649	"	0	0	1:3:6	5 mos. 5 days	159.26	111 482	700	1 038	.....	.....	.....	.....	57.8	.....	.....	.....	.....
	1 655	"	0	0	1:3:6	5 mos. 6 days	159.39	169 000	1 660	1 038	.....	.....	.....	.....	52.2	.....	.....	.....	.....
	1 652	Eight, ¾ in. square twisted bars	2.817	0	1:2:4	5 mos. 5 days	159.77	484 000	3 029	1 827	.....	63 500	.....	44 000	49.7	.....	.....	.....	.....
	1 653	Four, ¾ in. square twisted bars	1.416	0	1:3:6	5 mos. 5 days	158.76	306 700	1 923	1 033	.....	65 700	.....	64 000	65.1	.....	.....	.....	.....
	1 654	Eight, ¾ in. square twisted bars	2.83	0	1:3:6	5 mos. 6 days	159.01	493 000	3 100	1 033	.....	74 000	.....	73 900	71.6	.....	.....	.....	.....
	1 687	None	0	0	1:5:0	6 mos. 18 days	158.76	171 800	1 080	1 072	.....	.....	.....	.....	24.8	.....	.....	.....	.....

TABLE 1.—(*Continued.*)

Source.	Column designation.	Reinforcement: Kind.	Reinforcement: Vertical percentage.	Reinforcement: Lateral percentage.	Ratio of mix.	Age.	Area, in square inches.	Maximum load, in pounds.	Ultimate strength, in pounds per square inch.	Average cylinder test.	Stress in concrete.	Stress in longitudinal steel.	f_s (Equation (2?)).	$n f_c$.*	n.†	m.	Cylinders: Age.	Cylinders: Ultimate strength, in pounds per square inch.	Remarks.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
Report of Tests of Metals	1 718	Steel wire cage, 2 by 2 in. mesh, 0.104 in. in diameter	0.135	0.1416	1 : 2 : 4	5 mos. 8 days	113.1	263 538	2 330	1 827	2 279	36 500	.....	.....	16.8	0.46	.....	.....	.....
	1 722	Steel wire cage, 3 by 3 in. mesh, 0.104 in. in diameter	0.0978	0.0945	1 : 2 : 4	5 mos. 13 days	113.1	212 000	2 140	1 827	2 110	32 200	.....	.....	15.3	0.411	.....	.....	.....
	1 724	Steel wire cage, 3 by 3 in. circular wires, 0.145 in in diameter and vertical wires, 0.104 in. in diameter	0.0934	0.1833	1 : 2 : 4	5 mos. 13 days	113.1	318 000	2 810	1 827	2 767	47 140	.....	.....	17.0	0.354	.....	.....	.....
	1 720	Steel wire cage, 2 by 2 in. mesh, 0.104 in. in diameter	0.1426	0.1416	1 : 3 : 6	5 mos. 8 days	113.1	159 111	1 406	1 033	1 277	91 000	.....	.....	72.2	2.023	.....	.....	.....
	1 719		0.1426	0.1416	1 : 3 : 6	5 mos. 8 days	113.1	168 000	1 480	1 033	1 337	82 600	.....	.....	61.8	1.458	.....	.....	.....
	1 721		0.1426	0.1416	1 : 3 : 6	5 mos 8 days	113.1	188 500	1 623	1 033	1 513	78 160	.....	.....	51.6	0.885	.....	.....	.....
	1 723	Steel wire cage, 3 by 3 in. mesh. 0.104 in. in diameter	0.0978	0.0945	1 : 3 : 6	5 mos. 12 days	113.1	197 340	1 745	1 033	1 707	39 000	.....	.....	22.9	0.173	.....	.....	.....
	1 725	Steel wire cage, 3 by 3 in. mesh 0.145 in. in diameter, circular and 0.104 in. in diameter vertical	0.0978	0.1833	1 : 3 : 6	5 mos. 11 days	113.1	206 000	1 821	1 033	1 736	98 200	.....	.....	45.6	0.795	.....	.....	.....

TABLE 1.—(*Continued.*)

TABLE 1.—(*Continued.*)

Source. (1)	Column designation. (2)	Reinforcement. Kind. (3)	Vertical percentage. (4)	Lateral percentage. (5)	Ratio of mix. (6)	Age. (7)	Area, in square inches. (8)	Maximum load, in pounds. (9)	Ultimate strength, in pounds per square inch. (10)	Average cylinder test. (11)	Stress in concrete. (12)	Stress in longitudinal steel. (13)	f_s (Equation (27)). (14)	$n f_c$* (15)	n. (16)	m. (17)	Cylinders. Age. (18)	Cylinders. Ultimate strength, in pounds per square inch. (19)	Remarks. (20)
and Other Materials, Watertown Arsenal, 1906	1734	None	0	0	1:1:0	6 mos. 11 days	113.1	367 000	4 320	4 292	.....	.....	.....	.....	10.88	.....	.....	.....	.....
	1735	Twenty-five hoops 1.5 by 0.12 in. and four angles, 1 by 1 by ⅛ in.	1.158	1.8	1:1:0	6 mos. 12 days	77.63	508 000	6 470	4 292	5 840	60 000‡	.....	.....	14.23	2.35	.....	.....	Equation (23).
	1780		1.158	1.8	1:1:2	5 mos. 0 days	77.63	466 000	5 930	3 580	5 300	60 000‡	.....	.....	21.95	4.0	.....	.....	" "
	1736	None	0	0	1:4:0	6 mos. 8 days	84.62	88 851	1 050	1 408	.....	.....	.....	.....	30.9	.....	.....	.....	.....
	1737	Same as No. 1735	1.158	1.8	1:4:0	6 mos. 8 days	77.63	235 000	2 992	1 408	2 323	60 000‡	44 050	.....	80.0	.....	.....	.....	Equation (27).
	1738	" " " "	1.158	1.8	1:4:8	6 mos. 11 days	77.63	255 000	3 246	1 408	2 580	60 000‡	48 900	.....	96.9	.....	.....	.....	" "
	1731	None	0	0	1:2:4	5 mos. 16 days	84.95	120 000	1 413	1 827	.....	.....	.....	.....	12.15	.....	.....	.....	.....
	1740	Thirteen hoops 1.5 by 0.12 in.; spaced 8 in. c. to c.	0	0.9	1:2:4	6 mos. 6 days	78.54	188 900	2 044	1 827	2 405	.....	91 800	.....	49.0	5.96	.....	.....	Equation (27).
	1741	Twenty-five hoops 1.5 by 0.12 in.; spaced 4 in. c. to c.	0	1.8	1:2:4	6 mos. 5 days	78.54	290 000	2 693	1 827	3 610	.....	68 800	.....	43.77	4.6	.....	.....	" "
	1739	Forty-seven hoops 1.5 by 0.12 in.; spaced 2 in. c. to c.	0	3.6	1:2:4	6 mos. 6 days	78.54	445 000	5 665	1 827	5 665	.....	54 050	.....	.....	.....	.....	.....	" "

TABLE 1.—(*Continued*).

Source.	Column designation.	Reinforcement. Kind.	Vertical percentage.	Lateral percentage.	Ratio of mix.	Age.	Area, in square inches.	Maximum load, in pounds.	Ultimate strength, in pounds per square inch.	Average cylinder test.	Stress in concrete.	Stress in longitudinal steel.	f_s (Equation (27)).	$n f_c$.*	n.†	m.	Cylinders. Age.	Cylinders. Ultimate strength, in pounds per square inch.	Remarks.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
Report of Tests of Metals	1738	Thirteen hoops 1.5 by 0.12 in.; and 4 angles, 1 by 1 by ⅛ in.	1.158	0.90	1:2:4	6 mos. 7 days	77.63	256 300	3 262	1 827	2 597	60 000‡	.....	.....	39.5	4.0	.....	.....	.....
	1692	Twenty-five hoops	1.158	1.80	1:2:4	15 days	77.63	260 400	3 315	1 827	2 650	60 000‡	50 500	.....	.....	.....	.....	.....	Equation (27).
	1726	1.5 by 0.12 in. and four angles, 1 by 1 by ⅛ in.	1.158	1.80	1:2:4	5 mos. 12 days	77.63	349 000	4 343	1 827	3 795	60 000‡	72 300	.....	59.9	.....	.....	.....	" "
	1728		1.158	1.80	1:2:4	5 mos. 12 days	77.63	370 000	4 710	1 827	4 060	60 000‡	77 400	.....	43.1	.....	.....	.....	" "
	1746	Forty-eight hoops 1.04 by 0.06 in. and four angles, 1 by 1 by ⅛ in...	1.158	1.248	1:2:4	5 mos. 12 days	77.63	324 000	4 125	1 827	3 470	60 000‡	.....	.....	34.6	3.01	.....	.....	.....
	1744		1.158	1.248	1:2:4	6 mos. 0 days	77.63	335 200	4 267	1 827	3 615	60 000‡	.....	.....	30.92	2.62	.....	.....	.....
	1782	None.....	0	0	1:3:6	5 mos. 11 days	84.62	63 465	750	1 033	.....	.....	.....	.....	22.1	.....	.....	.....	.....
	1742	Twenty-five hoops 1.5 by 0.12 in.; spaced 4 in. c. to c.....	0	1.80	1:3:6	6 mos. 4 days	78.54	162 300	2 066	1 033	2 066	.....	39 400	.....	80.0	.....	.....	.....	Equation (27).
	1743		0	1.80	1:3:6	6 mos. 4 days	78.54	96 000	1 222	1 033	1 222	.....	23 300	.....	108.6	.....	.....	.....	" "
	1727	Twenty-five hoops 1.5 by 0.12 in. and 4 angles, 1 by 1 by ⅛ in.....	1.158	1.80	1:3:6	5 mos. 11 days	77.63	325 000	4 139	1 033	3 495	.....	66 600	.....	48.1	.....	.....	.....	" "
	1729		1.158	1.80	1:3:6	5 mos. 3 days	77.63	299 000	3 808	1 033	3 150	.....	60 000	.....	58.5	.....	.....	.....	" "
	1 747	48 hoops, 1.06 by 0.06 in. and 4 angles, 1 by 1 by ⅛ in.....	1.158	1.272	1:3:6	5 mos. 11 days	77.63	212 000	2 700	1 033	2 028	60 000‡	54 700	.....	41.35	.....	.....	.....	" "
	1 745		1.158	1.272	1:3:6	5 mos. 13 days	77.63	259 000	3 298	1 033	2 632	60 000‡	71 100	.....	66.9	.....	.....	.....	" "

TABLE 1.—(*Continued*.)

TABLE 1.—(*Continued.*)

Source.	Column designation.	Reinforcement. Kind.	Reinforcement. Vertical percentage.	Reinforcement. Lateral percentage.	Ratio of mix.	Age.	Area, in square inches.	Maximum load, in pounds.	Ultimate strength, in pounds per square inch.	Average cylinder test.	Stress in concrete.	Stress in longitudinal steel.	f_s (Equation (27)).	$n f_c$.*	n.	m.	Cylinders. Age.	Cylinders. Ultimate strength, in pounds per square inch.	Remarks.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
and Other Materials, Watertown Arsenal, 1907. ↑	1 809	None	0	0	1 : 0 : 1	8 mos. 19 days	144.6	514 000	3 555	4 292	.....	.....	.....	.....	15.02	.....	.....	.....	.....
	1 799	"	0	0	1 : 0 : 2	7 mos. 26 days	144.48	408 000	2 824	3 580	.....	.....	.....	.....	16.82	.....	.....	.....	.....
	1 794	"	0	0	1 : 0 : 3	7 mos. 23 days	144.96	281 000	1 938	2 177	.....	.....	.....	.....	19.57	.....	.....	.....	.....
	1 798	"	0	0	1 : 0 : 4	7 mos. 25 days	146.4	218 000	1 427	1 408	.....	.....	.....	.....	38.52	.....	.....	.....	.....
	1 797	"	0	0	1 : 0 : 5	7 mos. 24 days	144.48	161 400	1 117	1 072	.....	.....	.....	.....	55.5	.....	.....	.....	.....
	1 793	"	0	0	1 : 0 : 6	7 mos. 16 days	145.08	121 000	834	892	.....	.....	.....	.....	36.4	.....	.....	.....	.....
	1 796	"	0	0	1 : 0 : 7	7 mos. 17 days	144.72	108 540	750	800	.....	.....	.....	.....	29.85	.....	.....	.....	.....
	1 792	"	0	0	1 : 0 : 8	7 mos. 14 days	144.84	101 100	697	780	.....	.....	.....	.....	99.0	.....	.....	.....	.....
	1 812	"	0	0	1 : 0 : 2	9 mos. 0 days	155.38	800 000	5 149	3 580	.....	.....	.....	.....	.....	.....	.....	.....	Column did not fail.
	1 788	"	0	0	1 : 0 : 3	8 mos. 5 days	154.88	389 000	2 520	2 177	.....	.....	.....	.....	19.47	.....	.....	.....	.....
	1 791	"	0	0	1 : 0 : 4	8 mos. 4 days	155.0	170 500	1 100	1 408	.....	.....	.....	.....	51.6	.....	.....	.....	.....
	1 790	"	0	0	1 : 0 : 1	8 mos. 11 days	155.75	643 000	4 128	4 292	.....	.....	.....	.....	14.37	.....	.....	.....	.....
	1 813	"	0	0	1 : 0 : 1	9 mos. 9 days	155.87	792 000	5 081	4 292	.....	.....	.....	.....	9.97	.....	.....	.....	.....
	1 810	"	0	0	1 : 0 : 2	9 mos. 8 days	154.88	431 000	2 783	3 358	.....	.....	.....	.....	15.45	.....	.....	.....	.....

TABLE 1.—(*Continued.*)

Source.	Column designation.	Reinforcement. Kind.	Reinforcement. Vertical percentage.	Reinforcement. Lateral percentage.	Ratio of mix.	Age.	Area, in square inches.	Maximum load, in pounds.	Ultimate strength, in pounds per square inch.	Average cylinder test.	Stress in concrete.	Stress in longitudinal steel.	f_s (Equation (27)).	nf_c.*	n.†	m.	Cylinders. Age.	Cylinders. Ultimate strength, in pounds per square inch.	Remarks.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
Report of Tests of Metals	1 811	None	0	0	1 : 0 : 2	9 mos. 11 days	155.75	528 000	3 390	3 358	....	....	....	....	10.56	....	....	....	....
	1 789	"	0	0	1 : 0 : 3	8 mos. 8 days	156.12	807 500	1 970	2 177	....	....	....	....	24.79	....	....	....	....
	1 802	"	0	0	1 : 0 : 2	7 mos. 19 days	156.52	549 900	3 753	3 358	....	....	....	....	7.12	....	....	....	....
	1 801	"	0	0	1 : 0 : 4	7 mos. 14 days	145.44	275 300	1 893	1 408	....	....	....	....	27.05	....	....	....	....
	1 795	"	0	0	1 : 0 : 6	7 mos. 12 days	144.96	137 712	950	892	....	....	....	....	55.6	....	....	....	....
	1 800	"	0	0	1 : 0 : 8	7 mos. 12 days	144.96	131 100	904	780	....	....	....	....	38.7	....	....	....	....
	1 808	"	0	0	1 : 0 : 1	7 mos. 19 days	146.05	639 000	4 375	4 292	....	....	....	....	11.28	....	....	....	....
	1 807	"	0	0	1 : 0 : 2	7 mos. 20 days	145.08	409 000	2 819	3 358	....	....	....	....	14.22	....	....	....	....
	1 806	"	0	0	1 : 0 : 3	7 mos. 4 days	144.60	272 000	1 881	2 177	....	....	....	....	15.84	....	....	....	....
	1 805	"	0	0	1 : 1 : 1	7 mos. 10 days	145.92	514 000	3 522	3 590	....	....	....	....	14.78	....	....	....	....
	1 804	"	0	0	1 : 2 : 2	7 mos. 8 days	144.84	192 000	1 326	2 000	....	....	....	....	17.10	....	....	....	....
	1 803	"	0	0	1 : 3 : 3	7 mos. 5 days	145.08	121 000	834	834	....	....	....	....	29.10	....	....	....	....
	1 773	"	0	0	1 : 0 : 0	10 mos. 25 days	84.46	591 220	7 000	.....	....	....	....	....	10.0	....	....	....	Column did not fail.
	1 774	"	0	0	1 : 1.5 : 3	10 mos. 23 days	84.46	302 000	3 576	3 580	....	....	....	....	9.92	....	....	....	....

TABLE 1.—(*Continued.*)

TABLE 1.—(*Continued.*)

Source.	Column designation.	Reinforcement. Kind.	Reinforcement. Vertical percentage.	Reinforcement. Lateral percentage.	Ratio of mix.	Age.	Area, in square inches.	Maximum load, in pounds.	Ultimate strength, in pounds per square inch.	Average cylinder test.	Stress in concrete.	Stress in longitudinal steel.	f_c (Equation (27)).	nf_c.*	n.‖	m.	Cylinders. Age.	Cylinders. Ultimate strength, in pounds per square inch.	Remarks.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
	1 775	None	0	0	1 : 2 : 4	10 mos. 22 days	112.53	270 072	2 400	1 827	.....	.....	.....	.....	12.92	.....	.....	.....	.....
	1 750	48 hoops 1.04 by 0.06 in., 4 angles 1 by 1 by ⅛ in...	1.158	1.248	1 : 1 : 0	12 mos. 5 days	77.63	520 000	6 493	4 292	5 995	60 000	.....	.....	15.19	.....	.....	.....	.....
	1 748		1.158	1.248	1 : 1 : 2	12 mos. 3 days	77.63	479 000	3 580	5 470	5 470	60 000	.....	.....	16.05	.....	.....	.....	.....
	1 751		1.158	1.248	1 : 4 : 0	12 mos. 4 days	77.63	233 000	1 408	2 298	2 298	60 000	63 100	.....	67.6	.....	.....	.....	Equation (27).
	1 749		1.158	1.248	1 : 4 : 8	12 mos. 4 days	77.63	215 000	2 737	.....	2 066	60 000	.....	.....	64.7	.....	.....	.....	No. 1 : 4 : 8 plain concrete test.
	1 757	4 angles 1 by 1 by ⅛ in. held by two hoops 1.04 by 0.06 in.....	1.077	0	1 : 2 : 4	11 mos. 24 days	83.55	133 800	1 584	1 827	1 278	29 630	.....	.....	23.26	.....	.....	.....	Not as strong as plain concrete column.
	1 758		1.077	0	1 : 2 : 4	11 mos. 20 days	83.39	140 000	1 661	1 827	1 198	44 100	.....	.....	36.76	.....	.....	.....	
	1 756	4 angles 1 by 1 by ⅛ in., and 5 hoops 1.04 by 0.06 in....	1.075	0	1 : 2 : 4	11 mos. 25 days	83.71	135 392	1 600	1 827	1 179	37 300	.....	.....	49.2	.....	.....	.....	
	1 754	24 hoops 1.04 by 0.06 in., spaced 4 in. c. to c.....	0	0.624	1 : 2 : 4	11 mos. 23 days	78.54	186 000	2 367	1 827	2 368	.....	.....	.....	48.1	4.45	.....	.....	.....
	1 752	48 hoops 1.04 by 0.06 in., spaced 2 in. c. to c.....	0	1.248	1 : 2 : 4	12 mos. 0 days	78.54	262 000	3 335	1 827	3 335	.....	.....	.....	26.7	2.5	.....	.....	.....
	1 753		0	1.248	1 : 3 : 6	11 mos. 24 days	78.54	232 500	2 957	1 033	2 970	.....	.....	.....	66.5	3.7	.....	.....	.....
	1 765	25 hoops 1.5 by 0.12 in., 4 angles, 1 by 1 by ⅛ in....	1.158	1.80	1 : 3 : 6	14 mos. 0 days	77.63	288 000	3 667	1 033	3 012	60 000	89 200	.....	93.0	.....	.....	.....	Equation (27).

TABLE 1.—(*Continued.*)

Source. (1)	Column designation. (2)	Reinforcement. Kind. (3)	Vertical percentage. (4)	Lateral percentage. (5)	Ratio of mix. (6)	Age. (7)	Area, in square inches. (8)	Maximum load, in pounds. (9)	Ultimate strength, in pounds per square inch. (10)	Average cylinder test. (11)	Stress in concrete. (12)	Stress in longitudinal steel. (13)	f_c (Equation (27)). (14)	$n f_c$. (15)	n. (16)	m. (17)	Cylinders. Age. (18)	Cylinders. Ultimate strength, in pounds per square inch. (19)	Remarks. (20)
Watertown Arsenal, 1906	1 755	25 hoops 1.04 by 0.06 in., spaced, 4 in. c. to c.	0	0.624	1 : 3 : 6	12 mos. 0 days	78.54	71 600	912	1 033	912	.....	50 150	.....	21.3	.....	.....	.....	Equation (27).
	1 784	Four, ¾ in. square twisted bars.	1.444	0	1 : 2 : 4	17 mos. 21 days	155.99	358 777	2 300	1 827	1 765	38 850	.....	34 500	22.0	.....	.....	.....	.....
	1 785	Eight, ¾ in. square twisted bars.	2.896	0	1 : 2 : 4	17 mos. 22 days	155.37	574 869	3 700	1 827	2 130	56 400	.....	59 600	26.46	.....	.....	.....	.....
	1 776	¼ in. helix, pitch 2 in., and 4 rods 0.62 in. in diameter.	3.306	1.57	1 : 2 : 0	9 mos. 20 days	36.6	172 000	4 702	3 358	2 786	57 500	60 800	.....	20.63	.....	.....	.....	Equation (27).
	1 777	0.3 in. helix, pitch 1½ in., 4 rods, 0.62 in. in diameter.	2.11	2.20	1 : 2 : 2	10 mos. 0 days	57.4	435 000	7 580	2 000	6 780	37 920	105 800	.....	5.58	0.677	.....	.....	Strength of plain concrete assumed.
	1 778	5/16 in. helix, pitch 1 in. 6 rods, 0.62 in. in diameter.	2.15	2.962	1 : 2 : 2	10 mos. 14 days	84.1	850 000	10 450	2 000	6 430	60 000‡	74 000	.....	9.80	.....	.....	.....	Equation (27).
	1 715	Four Truscan bars each 0.76 in. in diameter. Eleven ⅜ in. wire hoops.	1.163	0	1 : 2 : 4	5 mos. 10 days	155.5	367 500	2 363	1 827	1 650	62 900	.....	39 200	38.1	.....	.....	.....	.....
	1 705	None.	0	0	1 : 3 : 6	*5 mos. 5 days	113.47	164 000	1 446	1 033	1 446	.....	.....	.....	21.1	.....	.....	.....	.....
	1 706	"	0	0	1 : 3 : 6	5 mos. 3 days	113.47	143 000	1 260	1 033	1 260	.....	.....	.....	17.2	.....	.....	.....	.....
	1 707	"	0	0	1 : 3 : 6	5 mos. 0 days	113.10	78 900	698	1 033	698	.....	.....	.....	30.8	.....	.....	.....	Cinder concrete.

TABLE 1.—(*Continued.*)

TABLE 1.—(*Continued.*)

Source.	Column designation.	Reinforcement. Kind.	Reinforcement. Vertical percentage.	Reinforcement. Lateral percentage.	Ratio of mix.	Age.	Area, in square inches.	Maximum load, in pounds.	Ultimate strength, in pounds per square inch.	Average cylinder test.	Stress in concrete.	Stress in longitudinal steel.	f_s (Equation (27))	$n f_c$.*	n.†	m.	Cylinders. Age.	Cylinders. Ultimate strength, in pounds per square inch.	Remarks.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
Am. Soc. C. E., Vol. LXXVIII (1915)	36	None	0	0	1:2:4	42 days	12.76	14 120	1 108	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	37	"	0	0	1:2:4	41 "	12.88	12 260	950	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	39	"	0	0	1:2:4	40 "	12.88	14 560	1 130	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	41	"	0	0	1:2:4	40 "	12.88	11 640	905	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	118	"	0	0	1:2:4	35 "	12.79	17 200	1 350	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
		Average	0	0	1:2:4	40 "	.....	.....	1 089	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	38	None	0	0	1:2:4	63 "	12.69	22 660	1 790	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	42	"	0	0	1:2:4	58 "	12.76	21 980	1 720	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	43	"	0	0	1:2:4	73 "	13.01	16 600	1 275	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	44	"	0	0	1:2:4	74 "	12.88	17 840	1 385	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	49	"	0	0	1:2:4	71 "	12.69	29 890	2 345	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	58	"	0	0	1:2:4	61 "	12.76	22 420	1 760	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	61	"	0	0	1:2:4	77 "	12.82	29 450	2 300	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
		Average	0	0	1:2:4	68 "	.....	.....	1 796	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	45	None	0	0	1:2:4	268 "	12.76	34 980	2 740	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	46	"	0	0	1:2:4	296 "	12.76	37 700	2 955	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	47	"	0	0	1:2:4	288 "	12.88	38 000	2 955	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	50*a*	"	0	0	1:2:4	340 "	12.76	37 000	2 900	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	53	"	0	0	1:2:4	295 "	12.69	39 570	3 120	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	67	"	0	0	1:2:4	323 "	12.69	31 300	2 470	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	76	"	0	0	1:2:4	376 "	12.76	28 000	2 200	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
		Average	0	0	1:2:4	313 "	.....	.....	2 763	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	48	None	0	0	1:2:4	414 "	12.82	45 800	3 570	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	51	"	0	0	1:2:4	434 "	12.82	46 300	3 600	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	52	"	0	0	1:2:4	440 "	12.82	35 000	2 725	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	54	"	0	0	1:2:4	441 "	12.88	40 700	3 160	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	62	"	0	0	1:2:4	434 "	12.76	36 350	2 850	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	79	"	0	0	1:2:4	416 "	12.95	28 060	2 170	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	80	"	0	0	1:2:4	417 "	12.88	29 000	2 250	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
		Average	0	0	1:2:4	428 "	.....	.....	2 903	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

TABLE 1.—(*Continued.*)

Source.	Column designation.	Reinforcement. Kind.	Vertical percentage.	Lateral percentage.	Ratio of mix.	Age.	Area, in square inches.	Maximum load, in pounds.	Ultimate strength, in pounds per square inch	Average cylinder test.	Stress in concrete.	Stress in longitudinal steel.	f_s (Equation (27))	nf_c*	n	m.	Cylinders. Age.	Cylinders. Ultimate strength, in pounds per square inch.	Remarks.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
of Steel," by C. G. Wrentmore, M. Am. Soc. C.E., *Transactions*,	110	0.062 in. wire helix, ¾ in. pitch	0	0.47	1:2:4	85 days	.....	23 700	2 650	1 490	....	.....	.....	.....	.....	.....	.....	.....	.....
	71		0	0.45	1:2:4	317 "	.....	274 500	2 773	.....	....	.....	.....	.....	.....	.....	.....	.....	.....
	74		0	0.44	1:2:4	370 "	.....	47 800	4 632	.....	....	.....	.....	.....	.....	.....	.....	.....	.....
	75	0.062 in. wire helix, 0.62 in. pitch	0	0.54	1:2:4	300 "	.....	40 500	4 091	.....	....	.....	.....	.....	.....	.....	.....	.....	.....
	78		0	0.54	1:2:4	281 "	.....	31 300	3 160	.....	....	.....	.....	.....	.....	.....	.....	.....	.....
	82		0	0.52	1:2:4	413 "	.....	33 700	3 265	.....	....	.....	.....	.....	.....	.....	.....	.....	.....
		Average of 71-82	0	0.50	1:2:4	336 "	.....	.....	3 584	2 833	....	.....	.....	.....	.....	.....	.....	.....	.....
	66	0.079 in. wire helix, ¾ in. pitch	0	0.72	1:2:4	74 "	.....	29 000	2 810	.....	....	.....	.....	.....	.....	.....	.....	.....	.....
	101	0.062 in. wire helix, ½ in. pitch	0	0.71	1:2:4	29 "	.....	22 000	2 460	.....	....	.....	.....	.....	.....	.....	.....	.....	.....
	107	0.079 in. wire helix, ¾ in. pitch	0	0.78	1:2:4	35 "	.....	28 000	3 130	.....	....	.....	.....	.....	.....	.....	.....	.....	.....
		Average	0	0.74	1:2:4	46 "	.....	.....	2 800	1 490	....	.....	.....	.....	.....	.....	.....	.....	.....
	60	0.079 in. wire helix, ¾ in. pitch	0	0.72	1:2:4	311 "	.....	38 200	3 702	....	....	.....	.....	.....	.....	.....	.....	.....	.....
	64		0	0.67	1:2:4	435 "	.....	41 500	3 520	....	....	.....	.....	.....	.....	.....	.....	.....	.....
	69		0	0.72	1:2:4	323 "	.....	31 350	3 040	.....	....	.....	.....	.....	.....	.....	.....	.....	.....
	72		0	0.72	1:2:4	388 "	.....	33 500	3 246	.....	....	.....	.....	.....	.....	.....	.....	.....	.....
	81	0.079 in. wire helix, 0.62 in. pitch	0	0.86	1:2:4	270 "	.....	36 250	3 560	.....	....	.....	.....	.....	.....	.....	.....	.....	.....
		Average	0	0.74	1:2:4	345 "	.....	.....	3 406	2 833	....	.....	.....	.....	.....	.....	.....	.....	.....
	59	0.015 in. wire helix, ¾ in. pitch	0	1.31	1:2:4	67 "	.....	40 250	4 134	.....	....	.....	.....	.....	.....	.....	.....	.....	.....
	98	0.079 in. wire helix ½ in. pitch	0	1.16	1:2:4	31 "	.....	31 500	3 520	.....	....	.....	.....	.....	.....	.....	.....	.....	.....
	104	0.105 in. wire helix, ¾ in. pitch	0	1.35	1:2:4	35 "	.....	24 500	2 740	.....	....	.....	.....	.....	.....	.....	.....	.....	.....
	105		0	1.35	1:2:4	49 "	.....	40 000	4 470	.....	....	.....	.....	.....	.....	.....	.....	.....	.....
	106		0	1.35	1:2:4	44 "	.....	34 800	3 890	.....	....	.....	.....	.....	.....	.....	.....	.....	.....
		Average	0	1.30	1:2:4	45 "	.....	.....	3 761	1 490	....	.....	.....	.....	.....	.....	.....	.....	.....
	63	0.105 in. wire helix, ¾ in. pitch	0	1.24	1:2:4	395 "	.....	37 500	3 520	.....	....	.....	.....	.....	.....	.....	.....	.....	.....
	65		0	1.26	1:2:4	316 "	.....	42 160	4 085	.....	....	.....	.....	.....	.....	.....	.....	.....	.....
	68		0	1.24	1:2:4	321 "	.....	36 570	3 420	.....	....	.....	.....	.....	.....	.....	.....	.....	.....
	73		0	1.26	1:2:4	428 "	.....	48 600	4 710	.....	....	.....	.....	.....	.....	.....	.....	.....	.....
		Average	0	1.25	1:2:4	365 "	.....	.....	3 934	2 833	....	.....	.....	.....	.....	.....	.....	.....	.....

TABLE 1.—(*Continued.*)

Source.	Column designation.	Reinforcement. Kind.	Vertical percentage.	Lateral percentage.	Ratio of mix.	Age.	Area, in square inches.	Maximum load, in pounds.	Ultimate strength, in pounds per square inch.	Average cylinder test.	Stress in concrete.	Stress in longitudinal steel.	f_c (Equation (27)).	$n f_c$.*	n.	m.	Cylinders. Age.	Cylinders. Ultimate strength, in pounds per square inch.	Remarks.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
Series of Tests on Concrete Columns Reinforced with a Spiral	56	0.079 in. wire helix,	0	1.68	1:2:4	58 days	.....	40 590	4 219	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	57	⅛ in. pitch.....	0	1.65	1:2:4	59 "	.....	39 000	3 939	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	94	0.062 in. wire helix,	0	1.41	1:2:4	35 "	.....	35 270	3 940	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
		¼ in. pitch.....																	
		Average.....	0	1.58	1:2:4	51 "	.....	.....	4 033	1 490	.....	.....	.....	.....	.....	.....	.....	.....	.....
	77	0.105 in. wire helix,	0	1.51	1:2:4	277 "	.....	42 700	4 138	2 833	.....	.....	.....	.....	.....	.....	.....	.....	.....
		0.62 in. pitch....																	
	89	0.079 in. wire helix,	0	2.31	1:2:4	35 "	.....	49 900	5 580	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
		¼ in. pitch																	
	96	0.105 in. wire helix,	0	2.02	1:2:4	35 "	.....	42 700	4 770	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	97	½ in. pitch.....	0	2.02	1:2:4	56 "	.....	51 580	5 760	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
		Average of 89, 97, 96.	0	2.11	1:2:4	42 "	.....	.....	5 370	1 490	.....	.....	.....	.....	.....	.....	.....	.....	.....
	70	0.079 in. wire helix,	0	2.32	1:2:4	393 "	.....	53 690	6 000	2 833	.....	.....	.....	.....	.....	.....	.....	.....	.....
		¼ in. pitch.....																	
	87	0.105 in. wire helix,	0	4.10	1:2:4	35 "	.....	71 800	7 965	1 490	.....	.....	.....	.....	.....	.....	.....	.....	.....
		¼ in. pitch.....																	
	10	0.105 in. wire helix,	0	1.87	1:2:4	28 "	.....	30 770	3 020	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
		½ in. pitch.....																	
	16	0.079 in. wire helix,	0	1.87	1:2:4	28 "	.....	45 550	4 740	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	17	¼ in. pitch.....	0	1.87	1:2:4	28 "	.....	51 100	5 310	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
		Average 10, 16, 17....	0	1.87	1:2:4	28 "	.....	.....	4 390	1 490	.....	.....	.....	.....	.....	.....	.....	.....	.....
	11	0.104 in. wire helix	0	1.25	1:2:4	28 "	.....	30 100	3 130	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
		¾ in. pitch.....																	
	18	0.079 in. wire helix,	0	1.26	1:2:4	28 "	.....	47 670	4 950	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
		⅛ in. pitch.....																	
		Average 11 and 18...	0	1.255	1:2:4	28 "	.....	.....	4 040	1 490	.....	.....	.....	.....	.....	.....	.....	.....	.....
	12	0.134 in. wire helix,	0	2.09	1:2:4	28 "	.....	32 300	3 360	1 490	.....	.....	.....	.....	.....	.....	.....	.....	.....
		¾ in. pitch.....																	
	13	0.165 in. wire helix,	0	2.30	1:2:4	28 "	.....	42 100	4 370	1 490	.....	.....	.....	.....	.....	.....	.....	.....	.....
		1 in. pitch.....																	
	14	0.165 in. wire helix,	0	1.54	1:2:4	28 "	.....	32 500	3 380	1 490	.....	.....	.....	.....	.....	.....	.....	.....	.....
		1½ in. pitch.....																	

TABLE 1.—(*Continued.*)

Source.	Column designation.	Reinforcement. Kind.	Reinforcement. Vertical percentage.	Reinforcement. Lateral percentage.	Ratio of mix.	Age.	Area, in square inches.	Maximum load, in pounds.	Ultimate strength, in pounds per square inch.	Average cylinder test.	Stress in concrete.	Stress in longitudinal steel.	f_c (Equation (27)).	nf_c.*	n.‖	m.	Cylinders. Age.	Cylinders. Ultimate strength, in pounds per square inch.	Remarks.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
" Report of a	15	0.079 in. wire helix, ¼ in. pitch.....	0	1.95	1:2:4	28 days	.....	52 800	5 490	1 490	.....	.....	.....	.....	.....	.....	.....	.....	.....
	19	0.079 in. wire helix,	0	0.97	1:2:4	28 "	.....	39 600	4 120	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	20	½ in. pitch.....	0	0.96	1:2:4	28 "	.....	34 200	3 560	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
		Average 19 and 20...	0	0.965	1:2:4	28 "	.....	.....	3 840	1 490	.....	.....	.....	.....	.....	.....	.....	.....	.....
	21	0.079 in. wire helix, ⅛ in. pitch.....	0	0.41	1:2:4	28 "	.....	49 000	5 090	1 490	.....	.....	.....	.....	.....	.....	.....	.....	.....
	22	0.079 in. wire helix, 0.70 in. pitch	0	0.69	1:2:4	28 "	.....	40 950	4 250	1 490	.....	.....	.....	.....	.....	.....	.....	.....	.....
	92	0.062 in. wire helix, ¼ in. pitch.....	0	1.41	1:2:4	34 "	.....	24 000	2 680	1 490	.....	.....	.....	.....	.....	.....	.....	.....	.....
	101	0.062 in. wire helix, ⅛ in. pitch.....	0	0.71	1:2:4	29 "	.....	22 000	2 460	1 490	.....	.....	.....	.....	.....	.....	.....	.....	.....
↓	113	None.....	0	0	1:2:4	35 "	.....	17 200	1 350	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

*The value in Column (15) was obtained from the formula, $f = f_c[1 + (n-1)p]$, in which, n was the unknown quantity, f_c being the value taken from the average cylinder tests or plain columns of the same mix. †This value is the average of all the cylinder tests of 1 : 2 : 4 concrete. ‡The value was assumed, the longitudinal steel having been stressed considerably beyond the elastic limit. §Observed from extensometer readings some time before failure. ‖The values shown in the columns for n (16) are taken from the observed values of the tests at failure by exterpolation whenever such data were given. No doubt these values are generally less than the real values, since the deformation becomes proportionately much greater at failure. Having given the value of n, the values in the column for m (17) were then calculated from Equation (21) wherein all the other values were assumed to be known. As may be seen some are less than those obtained under the assumption of constant volume. However, the volume of concrete under compression always decreases. In such cases these calculated values are therefore too small. In which case either n, ϕ, or the strength of the concrete was greater than the assumed value.

Equation (21), $v = \dfrac{f_c'}{1 - \dfrac{p'\,n\,\tan^2(45° + \frac{1}{2}\phi)}{2\,m + (m-1)\,p'n}}$; Equation (23), $v = \dfrac{v_c}{1 - \dfrac{2\,p'\,n}{m\,[2\,m + (m-)\,p'\,n]}}$; Equation (27), $v = \frac{1}{2} f_c\, p' \tan^2\left(45° + \frac{1}{2}\phi\right)$.

EVAPORATION ON UNITED STATES RECLAMATION PROJECTS

By Ivan E. Houk,* M. Am. Soc. C. E.

Synopsis

It is roughly estimated that on Federal irrigation projects alone, 1 000 000 acre-ft. of water is lost each year by evaporation from reservoirs. Much of this loss is unavoidable, but in some instances substantial economies can be effected if the nature and extent of the losses are thoroughly studied.

Reclamation Bureau engineers have been collecting and studying evaporation data for many years. Floating or land pans, sometimes both, have been maintained on projects where large storage reservoirs are necessary parts of the works, the primary object, of course, being to determine as accurately as possible the actual evaporation losses from the reservoir surfaces. Although variations in type of installation and equipment sometimes have been necessary in order to meet local conditions, the general design for land-pan installations during recent years has followed the specifications adopted by the United States Weather Bureau for its Class A stations. The Weather Bureau has cooperated with the Reclamation Bureau, in many instances, in furnishing equipment as well as in publishing daily and monthly records. Records published in the "Climatological Data for the United States", issued by the Weather Bureau, are mostly for land-pan stations, but a few floating-pan records are included therein. Daily and monthly records at several stations included in the tabulations accompanying this paper have not been published.

This paper summarizes the evaporation investigations of the Reclamation Bureau† to date, the more important features of which are briefly described as follows:

1.—Mean monthly evaporation records, with corresponding temperature, wind, and relative humidity data, whenever available, are given for all stations on Federal irrigation projects.

2.—Maximum and minimum values of monthly and annual evaporation are given for selected stations.

Note.—The subject of this paper is one of ten selected by the Special Committee on Irrigation Hydraulics for study and research. The paper was prepared in co-operation with E. B. Debler, M. Am. Soc. C. E., at the suggestion of the Committee in order to make available for general use much valuable data heretofore only found in the files of the U. S. Bureau of Reclamation. The Committee after studying the paper has recommended its publication in *Proceedings* in order to elicit discussion of the subject and through the discussion the possible presentation of other data of a similar nature not heretofore generally available. (See Progress Report of the Committee, *Proceedings*, Am. Soc. C. E., March, 1925, Society Affairs, p. 137.)

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† See, also, "Records of Evaporation Obtained at 23 Different Stations in Various Parts of the United States," *Engineering News*, June 16, 1910.

Equation (21), $v = \frac{p'n \tan^2(45° + \frac{1}{2}\phi)}{1 - \frac{2m + (m-1)p'n}{2m + (m-1)p'n}}$; Equation (23), $v = \frac{2p'n}{1 - \frac{m[2m + (m-)p'n]}{}}$

3.—The necessity for considering meteorological conditions in applying evaporation data is shown by a diagrammatic comparison of records (Fig. 4) at the two Yuma stations.

4.—The evaporation from a land pan, 14 in. deep and 3 ft. square, set in the ground with earth banked around the sides, was found to be only 72% of the evaporation from a Class A land pan, 10 in. deep and 4 ft. in diameter, set on the ground with no earth around the sides.

5.—The evaporation from a land pan on the ground at the edge of an alfalfa field was found to be only 62% of the evaporation on a tower 11 ft. above the ground.

6.—Evaporation from a floating Class A pan mounted on a raft was found to be 67% of the evaporation from a similar pan on land.

7.—Sloshing troubles in floating pans have been partly eliminated by installing a sheet-iron honeycomb in the lower part of the pan, thus dividing it into compartments.

8.—Measurements of ice evaporation at Saco, Mont., showed daily rates of from 0.010 to 0.070 in., the mean temperatures varying from about 9.6 to 31.1° Fahr.

Mean Monthly Records.—Records of mean monthly evaporation at all stations on Federal irrigation projects, whether maintained by the Bureau of Reclamation or by other agencies, are summarized in Table 1. In addition to the actual data, pertinent descriptive data regarding location, equipment, exposure, records, and parties maintaining the stations, are given in the table. Stations in the same section of the country have been kept together so far as possible. The locations of the stations are indicated on the key map, Fig. 1, by the numbers enclosed in circles, these numbers being the same as those noted above the station descriptions in Table 1.

In order to avoid useless distinctions, the name, Reclamation Bureau, has been used in Table 1 to designate the former Reclamation Service as well as the present organization. Acknowledgments for use of unpublished records are due the irrigation officials of the Office of Public Roads and Rural Engineering, United States Department of Agriculture; also the California-Oregon Power Company.

Mean monthly evaporation records, in inches, are given in Table 1, under "Descriptive Data". Following the evaporation records, wherever the data were available, are columns giving mean monthly temperatures, in degrees Fahrenheit, mean monthly wind velocities, in miles per hour, and mean monthly relative humidities, in percentages, these being the principal meteorological factors that affect evaporation rates. Unfortunately, relative humidity data were available in only one or two instances. Years of record are given inclusively in all cases. Sizes of pans, for all stations except Station No. 2, are noted by giving the vertical depth first, then the distance across, then the shape of the cross-section, as "19 in. by 3 ft., square."

Evaporation records, like most observational data are difficult to maintain continuously for long periods of time. Unforeseen interruptions occur despite

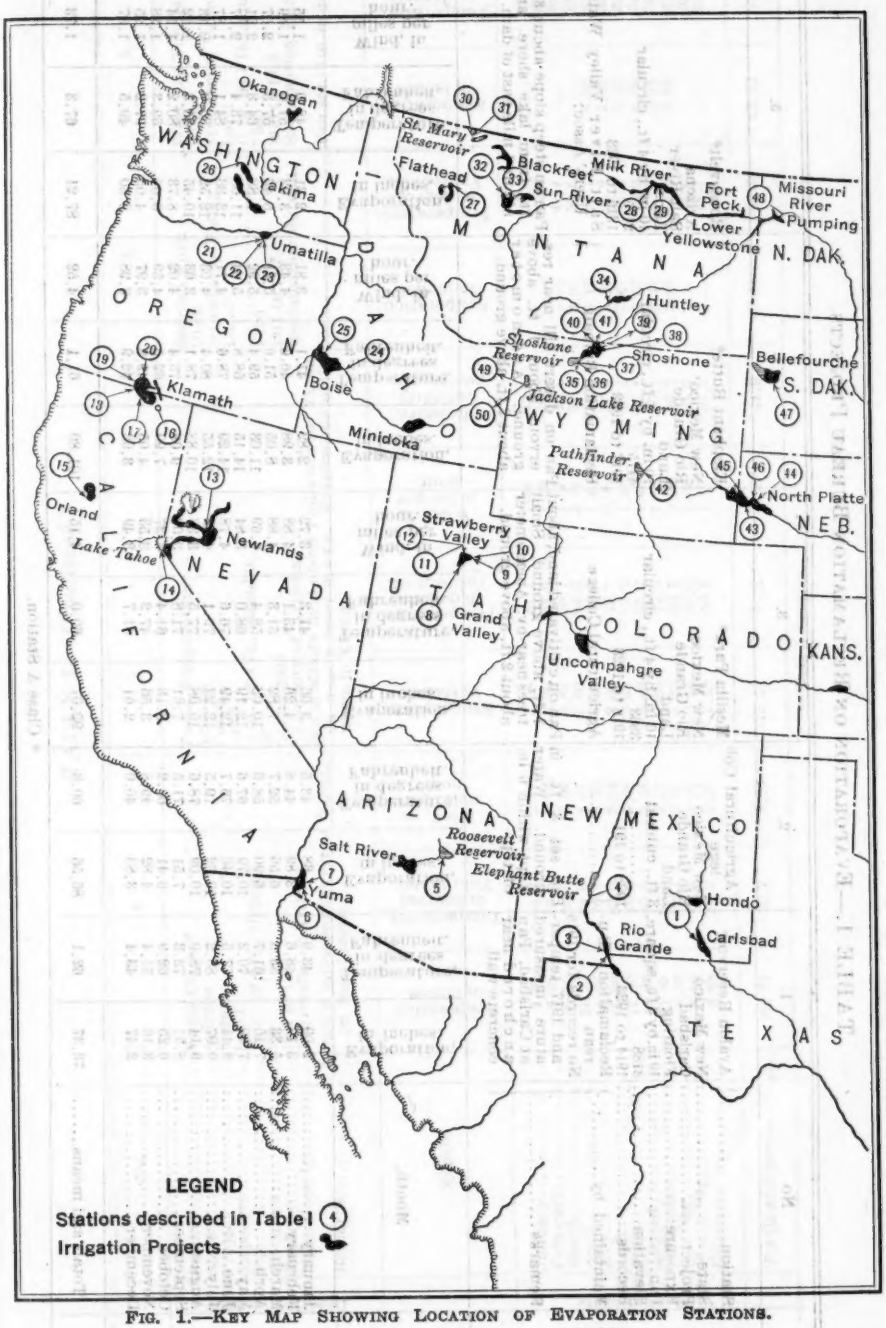

FIG. 1.—KEY MAP SHOWING LOCATION OF EVAPORATION STATIONS.

TABLE 1.—EVAPORATION ON RECLAMATION BUREAU PROJECTS.

	No.	1.		2.		3.			4.			5.		
Descriptive data	Station	Avalon Reservoir		Agricultural College		Mesilla Park*			Elephant Butte*			Roosevelt*		
	State	New Mexico		New Mexico		New Mexico			New Mexico			Arizona		
	Project	Carlsbad		Rio Grande		Rio Grande			Rio Grande			Salt River		
	Exposure	Floating		Land		Land			Land			Land		
	Pan	19 in. by 3 ft., square.		3 ft., cubical		10 in. by 4 ft., circular			10 in. by 4 ft., circular			10 in. by 4 ft., circular		
	Elevation	3188		.....		3863			4475			2175		
	Records	1914 to 1923		1908 to 191		1918 to 1923			1917 to 1923			1916 to 1923		
	Maintained by	Reclamation Bureau		.....		Agricultural College			Reclamation Bureau			Salt River Valley Water Users' Assoc.		
	Remarks	No records for 1916 and 1917 temperature measured at Carlsbad. Pan anchored near concrete wall.		Pan set 3 ft. in ground. Water kept within 6 in. of top.		Pan on cultivated land, about 1 ft. above ground. Small trees near-by. Anemometer about 2 ft. above ground.			Pan on desert hill near reservoir, about 1 ft. above ground. Anemometer about 2 ft. above ground.			Pan on steep slope about 300 ft. from lake shore. and about 1 mile east of dam.		
Mean records	Month.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Wind, in miles per hour.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Wind, in miles per hour.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Wind, in miles per hour.
	January	2.09	43.9	4.68	43.5	3.06	41.3	3.72	2.82	41.1	3.81	2.31	48.0	1.55
	February	3.10	48.6	3.89	44.8	4.26	45.1	4.88	3.98	46.6	4.43	3.03	51.6	1.69
	March	5.33	53.5	6.56	52.7	7.82	51.3	4.88	8.05	51.6	5.31	5.46	57.3	2.07
	April	7.15	61.7	6.90	58.9	10.03	58.4	6.69	11.03	59.4	5.92	7.88	64.3	2.21
	May	7.99	70.2	10.79	67.6	12.19	68.0	3.84	14.15	68.8	5.84	11.08	73.4	2.07
	June	8.41	77.5	10.46	76.7	12.49	76.6	4.78	14.29	78.6	4.71	13.36	84.1	1.87
	July	9.97	80.2	10.62	79.5	12.23	79.1	4.22	12.55	80.4	4.03	12.52	86.9	2.13
	August	9.14	78.9	10.09	78.6	10.08	77.5	3.36	10.88	78.1	3.63	10.45	85.3	1.62
	September	8.31	73.3	7.51	71.5	7.61	71.8	3.07	9.08	73.4	4.02	8.73	80.2	1.54
	October	6.25	62.9	6.41	60.2	6.13	61.4	3.37	7.90	62.5	4.59	6.04	69.3	1.38
	November	3.16	51.4	4.82	49.8	3.56	47.2	3.53	4.05	49.9	3.97	4.05	57.2	1.25
	December	2.47	43.4	3.83	40.0	2.61	41.7	3.49	3.02	42.9	3.98	2.30	49.5	1.37
	Totals and means	73.37	62.1	86.56	60.3	92.07	60.0	4.15	101.80	61.1	4.52	87.21	67.3	1.73

* Class A Station.

TABLE 1.—(*Continued.*)

	No.	6.			7.				8.		9.	
Descriptive data	Station	Yuma Citrus*			Yuma Evaporation*				Spanish Fork		West End Tunnel	
	State	Arizona			Arizona				Utah		Utah	
	Project	Yuma			Yuma				Strawberry Valley		Strawberry Valley	
	Exposure	Land			Land				Land		Land	
	Pan	10 in. by 4 ft., circular			10 in. by 4 ft., circular				.....		17 in. by 8 ft., square	
	Elevation	181			127				4 700		? 500	
	Records	1921 to 1923			1917 to 1923				1911 to 1915		1910 to 1915	
	Maintained by	University of Arizona			University of Arizona				Reclamation Bureau		Reclamation Bureau	
	Remarks	Pan 8 miles southwest of Yuma, on desert mesa			Pan in alfalfa field about 1 mile west of Yuma, on nearly level mesa. Relative humidity taken in Yuma				.....		Pan set 15 in. in ground	
Mean records	Month.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Wind, in miles per hour.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Wind, in miles per hour.	Relative humidity, percentage.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.
	January	4.23	53.2	2.41	3.08	51.1	1.44	47.0	....	30.0	....	21.8
	February	6.08	58.5	3.28	3.95	54.6	1.66	46.2	....	32.8	....	20.9
	March	8.71	61.8	3.52	5.82	58.7	1.87	40.1	....	41.4	....	31.4
	April	11.02	66.3	4.01	7.60	64.0	2.13	37.4	4.05	50.5	....	38.9
	May	14.32	75.9	3.70	8.25	69.9	1.40	38.6	5.17	57.1	4.55	47.0
	June	15.90	83.8	3.38	9.00	78.7	0.96	37.4	5.89	65.2	5.23	56.9
	July	17.53	84.9	4.05	10.42	87.0	1.31	46.4	7.67	72.6	5.20	61.8
	August	14.93	90.5	2.28	9.46	85.7	1.40	52.1	9.57	73.9	5.05	61.9
	September	11.85	84.8	2.57	7.30	79.9	1.18	50.1	6.41	62.9	3.44	52.9
	October	8.76	72.4	1.91	5.29	68.7	0.99	46.7	3.68	54.3	2.32	41.8
	November	5.50	61.3	1.99	3.16	57.6	1.02	45.5	....	41.0	....	32.6
	December	3.98	55.2	2.27	2.45	52.6	1.17	51.5	....	28.1	....	22.6
	Totals and means	122.81	70.7	3.03	75.81	67.4	1.38	44.9	42.44	50.8	25.79	40.9

*Class A Station.

TABLE 1.—(*Continued*).

	No.	10.		11.		12.			13.			14.		
Descriptive data	Station	East End Tunnel		Provo		Provo*			Fallon Experiment Farm			Lake Tahoe		
	State	Utah		Utah		Utah			Nevada			California		
	Project	Strawberry Valley		Strawberry Valley		Strawberry Valley			Newlands			Newlands		
	Exposure	Land		Land		Land			Land			Floating		
	Pan	17 in. by 3 ft., square		17 in. by 3 ft., square		10 in. by 4 ft., circular			2 by 6 ft., circular			10 in. by 4 ft., circular		
	Elevation	7600		4600		4600			3960			6230		
	Records	1905, 1914, and 1915		1910 to 1917		1918 to 1923			1908 to 1923			1916 to 1923		
	Maintained by	Reclamation Bureau		Reclamation Bureau		Reclamation Bureau			U. S. Department of Agriculture			Reclamation Bureau		
	Remarks	No temperature records for 1905. Pan set 15 in. in ground		Pan set 15 in. in ground		Pan in center of 0.38-acre field in irrigated district			Wind records began in 1912. Pan set 21 in. in ground			Pan submerged about 7 in. in river about 125 ft. from shore and 60 ft. from dam. Partly sheltered from wind		
Mean records	Month.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Evaporation in inches.	Temperature, in degrees Fahrenheit.	Wind, in miles per hour.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Wind, in miles per hour.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Wind, in miles per hour.
	January	....	13.4	....	25.9	....	27.9	....	1.12	29.9	2.75	1.58	26.0	3.04
	February	....	16.2	....	31.4	....	31.1	....	1.78	34.7	3.42	....	28.4	2.60
	March	....	24.1	....	41.3	....	40.2	1.80	4.03	43.3	4.47	3.37	30.5	2.96
	April	0.39	34.9	3.81	49.0	3.78	45.6	1.95	6.22	49.6	5.29	3.08	36.6	2.87
	May	1.32	44.2	4.29	55.6	6.06	56.4	1.62	8.28	56.7	4.34	3.75	44.4	2.51
	June	5.16	49.2	5.11	64.3	6.84	65.5	0.92	9.76	65.5	3.61	4.44	53.4	2.34
	July	5.52	57.2	5.14	70.8	7.12	71.7	0.84	10.65	73.4	2.81	5.69	60.5	2.50
	August	4.65	56.5	4.88	69.4	6.07	69.3	0.85	9.56	71.2	2.49	5.91	59.5	2.41
	September	2.60	47.2	3.42	60.3	4.18	61.4	0.84	6.40	61.5	2.60	4.61	53.5	2.28
	October	1.66	40.4	1.94	49.1	2.26	49.1	0.91	3.83	50.2	2.35	3.43	43.7	2.70
	November	0.21	29.5	0.87	39.3	0.94	38.4	0.73	1.91	39.9	2.37	2.62	35.3	2.74
	December	....	....	....	27.9	....	29.8	....	0.76	31.5	2.38	2.02	29.5	2.89
	Totals and means	21.51	34.4	29.46	48.7	37.25	48.9	1.16	64.30	50.6	3.24	40.50	41.8	2.65

*Class A Station.

TABLE 1.—(*Continued*).

	No.	15.		16.		17.		18.		19.		
Descriptive data	Station	East Park		Clear Lake		Adam's Boat Landing		Coppocks		A-Canal, Klamath Falls		
	State	California		California		Oregon		California		Oregon		
	Project	Orland		Klamath		Klamath		Klamath		Klamath		
	Exposure	Floating		Floating		Floating		Floating		.....		
	Pan	10 in. by 4 ft., circular		.....		10 in. by 4 ft., circular		10 in. by 4 ft., circular		.....		
	Elevation	1130 to 1200		4350		4070		4035		4100		
	Records	1911 to 1923		1911 to 1913		1913 to 1919		1916 to 1923		July, 1917, to June, 1918		
	Maintained by	Reclamation Bureau		Reclamation Bureau		Reclamation Bureau		Reclamation Bureau		Reclamation Bureau		
	Remarks	Anchored near reservoir shore. No wind obstructions		.....		Temperature taken at Merrill		Temperature taken at Merrill		Wind records for 1912 only		
Mean records	Month.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Wind, in miles per hour.
	January	1.13	43.3	....	....	....	....	....	....	....	35.0	5.24
	February	1.64	46.3	....	....	....	....	....	....	....	32.8	5.89
	March	2.79	50.0	....	....	....	....	....	....	....	39.4	7.92
	April	4.75	55.2	2.04	42.4	....	....	....	....	....	45.0	7.69
	May	7.22	62.5	3.48	49.7	5.85	50.6	6.32	49.7	5.17	51.7	7.80
	June	8.87	70.6	5.60	58.4	6.34	58.1	7.69	58.4	6.61	68.2	7.62
	July	10.54	76.5	7.68	66.0	7.71	63.6	7.61	64.1	9.08	72.2	7.05
	August	9.85	74.7	7.80	64.4	6.49	63.3	6.67	62.4	7.55	70.0	5.90
	September	7.21	68.4	5.88	51.8	4.48	56.9	4.47	57.6	4.61	60.2	6.32
	October	4.47	58.9	3.60	43.3	3.05	47.8	2.86	48.5	....	55.9	6.86
	November	2.38	49.3	1.44	36.9	....	....	1.00	....	....	40.6	5.03
	December	1.54	42.8	....	....	....	....	....	....	....	36.7	5.78
	Totals and means	62.39	58.2	37.52	34.4	33.92	56.7	36.62	56.8	33.02	50.6	6.59

TABLE 1.—(*Continued.*)

	No.	20.		21.			22.			23.		
Descriptive data	Station	Klamath Falls		Cold Springs			Cold Springs			Hermiston Experiment Farm		
	State	Oregon		Oregon			Oregon			Oregon		
	Project	Klamath		Umatilla			Umatilla			Umatilla		
	Exposure	Floating		Floating			Land			Land		
	Pan	22 by 42 in., circular		.....			10 in. by 4 ft., circular			2 by 6 ft., circular		
	Elevation	4100		600			623			450		
	Records	1921 to 1924		1909 to 1913			1914 to 1923			1912 to 1923		
	Maintained by	Calif.—Ore. Power Co.		Reclamation Bureau			Reclamation Bureau			U. S. Department of Agriculture		
	Remarks	In river about 500 ft. below main canal intake		Temperature taken at Hermiston			Temperature taken at Hermiston. Anemometer 17 ft. above ground. Pan near edge of reservoir resting on ground. Some brush and willows near-by			Anemometer 8 ft. above ground		
Mean records	Month.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Wind, in miles per hour.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Wind, in miles per hour.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Wind, in miles per hour.
	January	3.07	27.0	1.01	30.4	5.69	1.13	31.6	5.86	1.28	31.2	2.68
	February	2.52	32.8	1.12	35.7	6.42	0.94	36.5	6.02	0.90	34.8	2.53
	March	1.82	37.5	2.72	44.1	8.40	3.20	45.6	8.49	2.20	43.8	4.13
	April	2.82	44.8	5.86	52.7	9.93	4 21	52.2	8.62	3.96	51.1	4.34
	May	4.78	55.0	7.01	61.1	9.25	6.47	58.9	8.57	5.41	58.7	3.68
	June	6.51	61.5	8.90	65.6	10.29	7.90	67.2	8.65	7.22	67.2	3.72
	July	8.55	69.0	12.44	72.5	9.14	9.13	73.2	8.18	8.36	73.8	3.39
	August	7.49	66.0	9.40	69.3	7.93	8.29	71.6	6.71	6.63	72.6	2.45
	September	6.10	59.8	6.87	61.8	7.93	5.44	62.3	6.70	4.42	62.3	2.31
	October	4.33	49.0	3.80	50.0	6.57	3.23	52.1	5.63	2.49	51.4	2.15
	November	2.96	39.7	1.75	43.6	7.00	1.37	40.2	5.73	0.84	40.3	1.71
	December	....	30.0	1.08	32.9	6.07	0.71	31.3	5.72	0.27	32.0	2.34
	Totals and means	50.90	47.7	61.96	51.6	7.88	52.02	51.9	7.07	43.98	51.6	2.95

TABLE 1.—(*Continued*).

	No.	24.			25.				26.			27.	
Descriptive data	Station	Arrowrock*			Deer Flat*				Lake Kachess*			Ninepipe Reservoir	
	State	Idaho			Idaho				Washington			Montana	
	Project	Boise			Boise				Yakima			Flathead	
	Exposure	Land			Land				Land			Floating and land	
	Pan	10 in. by 4 ft., circular			10 in. by 4 ft., circular				10 in. by 4 ft., circular			10 in. by 5 ft., circular	
	Elevation	3220			2510				2270			3003	
	Records	1916 to 1923			1916 to 1923				1917 to 1923			1914 to 1923	
	Maintained by	Reclamation Bureau			Reclamation Bureau				Reclamation Bureau			Reclamation Bureau	
	Remarks	Pan on bench in hillside, about 30 ft. above maximum water surface in reservoir			Pan on farm lane, surrounded by irrigated fields, about 800 ft. west of lower embankment. Relative humidity data taken at Boise. Anemometer about 2 ft. above ground				Pan about 100 yd. from high-water line. Watchman's house, and a few trees near-by			Project now under Indian Service. Temperature taken at St. Ignatius. Pan moved several times	
Mean records	Month.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Wind, in miles per hour.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Wind, in miles per hour.	Relative humidity, percentage.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Wind, in miles per hour.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.
	January	....	26.6	....	....	26.3	2.62	73.5	....	27.8	....	....	23.8
	February	....	29.8	....	....	31.2	2.72	69.4	....	28.1	....	....	26.2
	March	....	37.6	....	....	41.3	5.00	59.0	....	33.5	....	....	36.0
	April	3.45	46.7	1.00	5.52	49.7	4.92	54.9	....	40.2	....	....	44.7
	May	5.76	55.4	1.04	5.65	57.7	3.47	52.8	3.30	47.2	2.41	4.88	51.6
	June	7.62	65.7	0.88	6.81	65.1	1.98	48.3	4.90	56.1	2.26	5.76	59.7
	July	10.19	74.9	0.87	7.99	73.0	1.31	37.8	6.11	63.5	2.28	6.03	66.7
	August	9.15	72.7	0.83	7.16	70.6	0.97	39.4	4.40	63.3	1.92	6.27	65.5
	September	5.79	63.4	0.85	4.95	59.5	1.44	47.0	2.14	55.4	1.36	3.97	55.9
	October	2.37	50.0	0.79	2.89	50.2	1.90	58.9	....	46.5	....	1.62	46.3
	November	....	37.7	....	1.09	38.7	2.34	66.0	....	35.4	....	....	33.7
	December	....	27.8	....	0.85	31.7	3.26	74.2	....	28.7	....	....	25.3
	Totals and means	44.33	49.0	0.89	42.91	49.6	2.66	56.8	20.85	43.8	2.05	28.53	44.6

*Class A Station.

TABLE 1.—(*Continued*).

	No.	28.		29.					30.			31.	
Descriptive data	Station	Nelson Reservoir		Nelson Reservoir					Sherburne Reservoir			Sherburne Reservoir	
	State	Montana		Montana					Montana			Montana	
	Project	Milk River		Milk River					Milk River			Milk River	
	Exposure	Floating		Floating and land					.....			Land	
	Pan	18 in. by 3 ft., square		10 in. by 4 ft., circular					.....			10 in. by 4 ft., circular	
	Elevation	2215		2215					4865			4865	
	Records	1917 to 1920		1921 to 1923					1915 to 1918			1922 to 1923	
	Maintained by	Reclamation Bureau		Reclamation Bureau					Reclamation Bureau			Reclamation Bureau	
	Remarks	Temperature taken at Malta. Pan on raft in lake, partly submerged		Wind records began in 1922. Floating pan anchored about 350 yd. from shore. Land pan on small knoll 100 yd. from shore. No wind obstructions					No wind records for 1918			Pan about 1 000 ft. from shore. No wind obstructions	
Mean records	Month.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Evaporation, in inches.			Temperature, in degrees Fahrenheit.	Wind, in miles per hour.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Wind, in miles per hour.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.
				Floating.	Land.	Ratio.							
	January	....	....	....	....	....	....	....	....	16.4	8.02	....	....
	February	....	....	....	....	....	....	....	....	14.8	7.92	....	....
	March	....	....	....	....	....	....	....	....	25.3	11.08	....	....
	April	....	....	....	....	....	....	....	....	32.9	4.97	....	....
	May	5.52	57.8	5.26	8.82	0.60	54.5	6.64	3.75	42.8	7.23	....	47.8
	June	6.26	66.9	6.37	9.57	0.67	65.9	5.80	5.56	53.6	6.93	4.90	55.4
	July	7.94	72.1	7.82	11.18	0.70	69.6	4.65	6.46	60.8	6.63	5.16	59.8
	August	8.20	69.5	8.67	11.36	0.76	64.2	4.69	4.97	57.4	5.43	4.86	59.4
	September	4.58	57.6	5.31	7.70	0.64	58.7	5.01	2.92	50.5	7.87	3.70	53.
	October	3.27	42.1	2.78	4.26	0.74	47.9	5.32	3.08	39.9	13.05	....	45.0
	November	....	....	....	....	....	34.9	....	....	32.5	13.53	....	....
	December	....	....	....	....	....	19.6	....	....	16.6	11.87	....	....
	Totals and means	35.77	61.0	36.21	52.89	0.68	51.9	5.35	26.74	37.0	8.71	18.62	53.4

TABLE 1.—*(Continued).*

	Number.	32.			33.			34.			35.	
Descriptive data.	Station	Sun River Diversion			Willow Creek			Osborn Experiment Farm			Shoshone Dam	
	State	Montana			Montana			Montana			Wyoming	
	Project	Sun River			Sun River			Huntley			Shoshone	
	Exposure	Floating			Floating			Land			Land	
	Pan	10 in. by 4 ft., circular			10 in. by 4 ft., circular			2 by 6 ft., circular			10 in. by 4 ft., circular	
	Elevation	4474			4130			3000			5390	
	Records	1919 to 1923			1919 to 1923			1911 to 1923			1915 to 1924	
	Maintained by	Reclamation Bureau			Reclamation Bureau			U. S. Department of Agriculture			Reclamation Bureau	
	Remarks	Some of the earlier records are questionable. Pan near north bank. No wind obstructions. Sides of pan partly submerged			Pan about 200 ft. from dam, sides partly submerged			Temperature taken at Huntley, 1911 to 1915; at Osborn, January, February, and March, 1919; and at Ballantine since March, 1919. Pan set 22 in. in ground, in open field			Pan on ground 1916 to 1918; 6 in. in ground in 1915; on roof, 1919 to 1922; and on ground, 1923 and 1924	
Mean records	Month.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Wind, in miles per hour.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Wind, in miles per hour.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Wind, in miles per hour.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.
	January	....	26.2	....	....	26.1	....	....	21.2	5.42	0.87	26.2
	February	....	22.6	....	....	23.0	....	....	23.0	4.68	1.25	29.6
	March	....	29.0	....	....	30.8	....	....	34.4	4.79	1.62	34.5
	April	5.69	39.0	....	3.93	39.4	9.71	3.33	45.5	5.20	3.44	42.5
	May	6.54	48.9	9.07	4.38	50.2	7.50	4.72	54.5	4.96	4.85	51.0
	June	4.95	56.5	8.23	5.45	60.6	6.35	6.01	64.9	3.70	7.46	62.0
	July	8.23	67.7	7.99	7.08	67.7	5.61	7.49	71.2	3.25	8.22	69.7
	August	6.01	63.4	8.15	7.83	66.0	5.42	6.80	69.1	3.20	7.88	68.2
	September	4.19	53.7	10.97	4.70	57.8	6.57	4.50	58.7	3.54	5.43	59.2
	October	2.79	43.4	12.83	3.87	45.8	6.57	....	46.9	3.88	2.97	48.5
	November	....	31.6	....	....	33.4	....	....	34.4	4.06	0.50	38.4
	December	....	23.8	....	....	24.0	....	....	24.8	4.60	....	28.3
	Totals and means	38.40	42.2	9.46	37.24	43.7	6.82	32.85	45.7	4.27	44.49	46.5

TABLE 1.—(*Continued.*)

	No.	36.		37.		38.		39.		40.	
Descriptive data	Station	Shoshone Dam		Shoshone Dam		Powell		Powell		Ralston	
	State	Wyoming		Wyoming		Wyoming		Wyoming		Wyoming	
	Project	Shoshone		Shoshone		Shoshone		Shoshone		Shoshone	
	Exposure	Land		Land		Land		Land		Land	
	Pan	10 in. by 4 ft., circular		10 in. by 4 ft., circular		8 in. by 3 ft., circular		4 ft., circular		3 ft., circular	
	Elevation	5390		5390		4390		4390		4600	
	Records	1915		1919 to 1922		1911 to 1919		1911 to 1919		1911 to 1916	
	Maintained by	Reclamation Bureau		Reclamation Bureau		Reclamation Bureau		Reclamation Bureau		Reclamation Bureau	
	Remarks	Pan set 6 in. in ground. These values included in No. 35		Pan on shed roof 10 ft. above ground. These values included in No. 35		Pan on tower 11 ft. above ground		Pan on edge of alfalfa field		Temperature taken at Powell. Pan on bank of reservoir. No wind obstructions	
Mean records	Month.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.
	January	0.87	29.0	....	29.8	....	17.6	....	17.6	....	17.9
	February	1.25	35.0	....	28.4	....	22.8	....	22.8	....	23.9
	March	1.62	34.6	....	35.3	1.20	32.7	1.01	32.7	....	32.8
	April	4.00	52.6	....	41.4	7.52	44.2	3.99	44.2	....	46.1
	May	3.94	49.2	5.44	52.6	7.68	52.2	5.74	52.2	6.00	52.4
	June	5.19	55.8	8.48	64.6	10.21	62.8	6.55	62.8	8.42	62.1
	July	6.00	62.4	8.79	70.3	10.34	68.2	6.49	68.2	9.23	67.5
	August	4.63	58.2	8.91	70.2	9.49	66.3	6.01	66.3	8.71	66.4
	September	2.63	55.0	7.22	60.3	6.30	56.7	3.29	56.7	5.87	56.1
	October	3.19	54.3	3.16	48.9	3.88	43.1	1.94	43.1	3.00	43.7
	November	0.50	39.2	....	35.9	....	32.3	....	32.3	....	30.0
	December	....	31.2	....	28.4	....	20.1	....	20.1	....	19.8
	Totals and means	33.82	47.2	42.00	47.2	56.62	43.2	35.02	43.2	41.23	43.5

TABLE 1.—(*Continued.*)

	No.	41.		42.		43.			44.			45.		
Descriptive data	Station	Ralston		Pathfinder Reservoir		Mitchell Experiment Farm			Lake Minatare			Dutch Flats Camp		
	State	Wyoming		Wyoming		Nebraska			Nebraska			Nebraska		
	Project	Shoshone		North Platte		North Platte			North Platte			North Platte		
	Exposure	Floating		Land		Land			Land			Land		
	Pan	3 ft., circular		10 in. by 4 ft., circular		2 by 6 ft., circular			10 in. by 4 ft., circular			10 in. by 4 ft., circular		
	Elevation	4590		5900		4080			4120			4110		
	Records	1911 to 1914		1914 to 1923		1911 to 1923			1918 and 1919			1909		
	Maintained by	Reclamation Bureau		Reclamation Bureau		U. S. Department of Agriculture			Reclamation Bureau			Reclamation Bureau		
	Remarks	Temperature taken at Powell. Pan floated on reservoir		Pan set on concrete foundation, sides exposed. No wind obstructions. About 1 000 ft. from reservoir		Pan set 22 in. in ground. In open cultivated field prior to 1922, when it was moved ¼ mile east. Now partly sheltered from wind			Pan on wood foundation just above ground, partly sheltered from wind; 800 ft. from shore			Pan on wood foundation just above ground, partly sheltered from wind		
Mean records	Month.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Wind, in miles per hour.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Wind, in miles per hour.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Wind, in miles per hour.
	January	....	21.2	....	22.8	....	24.3	6.55	....	24.3	9.33	....	....	....
	February	....	22.1	....	24.4	....	25.3	6.26	....	25.8	8.69	....	....	....
	March	....	31.3	....	32.9	....	34.5	7.50	....	37.7	9.37	....	....	....
	April	....	44.7	....	40.2	4.13	43.6	8.07	4.13	41.2	12.57	4.50	40.6	10.28
	May	4.00	53.7	6.60	51.2	6.23	53.9	7.76	7.68	55.5	9.17	6.25	50.0	12.16
	June	4.48	64.2	9.60	62.7	7.18	65.5	5.92	9.12	67.2	8.08	8.05	59.5	7.96
	July	4.68	67.7	11.03	70.5	8.02	71.5	4.36	9.65	73.9	7.74	10.95	69.2	6.37
	August	9.43	66.4	9.85	67.9	6.95	69.1	3.91	8.83	72.2	7.41	9.89	71.8	4.34
	September	4.74	56.9	7.40	59.0	5.24	59.8	4.29	4.48	61.4	5.61	7.44	60.1	6.29
	October	....	43.3	4.16	46.7	2.57	47.5	5.07	2.72	47.2	6.97	5.59	51.6	8.71
	November	....	34.4	....	35.4	....	35.5	5.06	....	30.0	7.97	....	....	....
	December	....	20.2	....	24.3	....	24.4	5.53	....	24.6	7.45	....	....	....
	Totals and means.	27.33	43.8	48.64	44.8	40.32	46.2	5.86	46.61	46.7	8.36	52.17	57.5	8.01

TABLE 1.—(*Continued*).

	No.	46.			47.			48.			49.		50.	
Descriptive data	Station	Sunflower Camp			Newell Experiment Farm			Williston Experiment Station			Jackson Lake		Jackson Lake	
	State	Nebraska			South Dakota			North Dakota			Wyoming		Wyoming	
	Project	North Platte			Belle Fourche			North Dakota Pumping Project			Minidoka		Minidoka	
	Exposure	Land			Land			.....			Land		Land	
	Pan	10 in. by 4 ft., circular			2 by 6 ft., circular			.....			14 in. by 3 ft., square		10 in. by 4 ft., circular	
	Elevation	4070			2880			.....			6770		6770	
	Records	1910 to 1917			1908 to 1923			1918 and 1919			1922 to 1924		1924	
	Maintained by	Reclamation Bureau			U. S. Dept. of Agriculture.			.....			Reclamation Bureau		Reclamation Bureau	
	Remarks	Pan on wood foundation just above ground, partly sheltered from wind			Pan set 21 in. in ground; surrounded by cultivated fields. Anemometer 2 ft. above ground			.....			Pan set in ground, earth banked around sides to water surface. Water kept 12 in. deep		Pan set on ground, no earth around sides	
Mean records	Month.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Wind, in miles per hour.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Wind, in miles per hour.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Wind, in miles per hour.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.	Evaporation, in inches.	Temperature, in degrees Fahrenheit.
	January	0.92	22.9	8.57	....	16.8	7.20	....	12.3	8.55	....	....	....	....
	February	0.84	25.9	7.32	....	18.4	6.55	....	4.2	7.80	....	....	....	....
	March	2.06	31.8	9.46	....	30.9	7.50	....	26.0	8.37	....	....	....	....
	April	4.48	47.1	10.25	3.99	43.8	8.05	4.06	42.8	10.60	....	....	....	....
	May	6.35	53.5	9.60	5.57	53.6	8.26	5.98	54.0	10.09	....	....	....	....
	June	7.94	64.4	8.96	6.88	65.6	6.67	7.21	67.3	10.40	5.49	51.7	....	51.2
	July	8.21	70.9	6.66	8.06	71.8	5.43	8.57	68.6	8.34	5.61	58.0	7.56	56.8
	August	6.78	68.4	6.53	7.02	69 2	5.08	6.42	67.7	8.29	4.75	56.5	8.60	54.1
	September	5.39	59.7	6.63	5.03	59.4	5.58	3.73	55.4	8.15	3.11	49.0	4.90	47.2
	October	3.11	47.7	6.95	....	45.9	6.70	....	39.4	8.03	....	....	....	....
	November	1.71	38.6	6.96	....	33.0	6.00	....	22.1	8.17	....	....	....	....
	December	....	23.2	6.90	....	20.1	8.40	....	16.0	7.65	....	....	....	....
	Totals and means	47.79	46.2	7.90	36.55	44.0	6.78	35.97	39.6	8.70	18.96	....	21.06	....

the best laid plans. Animals break through the fence and drink the water, waves splash over the edges of the pan, the pan develops a leak, or some other happening of similar consequence vitiates the records. Although the data for some of the stations, principally Class A installations, are practically continuous, records at others are not. As a general rule, interruptions are less frequent at land pans than at floating pans. They are also less frequent at the warmer stations where freezing seldom or never occurs. At the northern stations it is, of course, necessary to discontinue the records during the winter months. Temperature, wind, and relative humidity records are seldom interrupted, except when the stations are dismantled for the winter. Sometimes these records are maintained during the cold weather and only the evaporation observations discontinued.

In compiling the data in Table 1, questionable records were discarded. Records were not interpolated except in cases where only a few days' observations were missing. Ordinarily, if more than a week's data were missing in any one month the remaining records for that month were disregarded in calculating the averages. Consequently, the mean monthly values for a given station may not be entirely comparable as regards length of record. To illustrate, the mean evaporation for June may be calculated from five records whereas the mean evaporation for July may be based on six, seven, or some other number of observations.

Range in Evaporation.—The range in annual evaporation at selected stations is indicated by the data in Table 2. Maximum and minimum records, their differences, and the years in which they occurred, are given for the stations that had the greater numbers of full years of record. Quantities are given in inches, also in percentages of mean values. It will be noted that the total range varied from 8.55 in. at the East Park Reservoir to 25.47 in. at Cold Springs, the corresponding percentages being 13.7 and 49.0, respectively.

Similar data showing variations of monthly evaporation at the floating-pan station at the East Park Reservoir and at the land-pan station at Elephant Butte are given in Table 3, these stations having been chosen because their records contain practically no interruptions, thanks to the efficient supervision of R. C. E. Weber and L. M. Lawson, M. Am. Soc. C. E., Project Superintendents. The difference between maximum and minimum records for individual months vary from 1.14 to 4.20 in. at East Park, and from 2.34 to 4.40 in. at Elephant Butte. Fig. 2 is a view of the floating pan at East Park Reservoir. Fig. 3 shows the land-pan station at Mesilla Park, Rio Grande Project, maintained by the New Mexico Agricultural College. In compiling the data it was noted that records of daily evaporation as great as 0.8 in. have occurred at the Yuma and Roosevelt Stations.

Effect of Meteorological Conditions.—An unusual example of the variations in evaporation caused by variations in meteorological conditions is furnished by the records at two Yuma Stations. These are both Class A installations, maintained by the University of Arizona. Records are published by the U. S. Weather Bureau. The stations are less than 10 miles apart, and the mean annual temperatures differ by only 3.3 degrees. Consequently, one would expect the records to be quite similar. However, the annual evaporation

TABLE 2.—RANGE IN ANNUAL EVAPORATION AT SELECTED STATIONS.

Station.		Full years of record.	Mean annual evaporation, in inches.	Maximum Annual Evaporation.			Minimum Annual Evaporation.			Total Range.	
				Year.	Amount:		Year.	Amount:		In inches.	In percentage of mean.
No.	Name.				In inches.	In percentage of mean.		In inches.	In percentage of mean.		
1	Avalon Reservoir	6	73.37	1922	82.51	112.5	1921	65.75	89.6	16.76	22.9
3	Mesilla Park	5	92.07	1922	98.03	106.5	1923	84.74	92.0	13.29	14.5
4	Elephant Butte	7	101.80	1917	109.70	107.8	1919	90.19	88.6	19.51	19.2
5	Roosevelt	6	87.21	1917	94.64	108.4	1920	82.47	94.6	12.17	13.8
6	Yuma Citrus	3	122.81	1921	135.70	110.5	1923	113.69	92.6	22.01	17.9
7	Yuma Evaporation	5	75.81	1918	88.00	116.1	1920	70.31	92.7	17.69	23.4
13	Fallon Experiment Farm	8	64.30	1914	72.80	113.2	1913	60.94	94.8	11.86	18.4
15	East Park Reservoir	12	62.39	1923	67.01	107.4	1915	58.46	93.7	8.55	13.7
22	Cold Springs	6	52.02	1914	61.99	119.2	1923	36.52	70.2	25.47	49.0

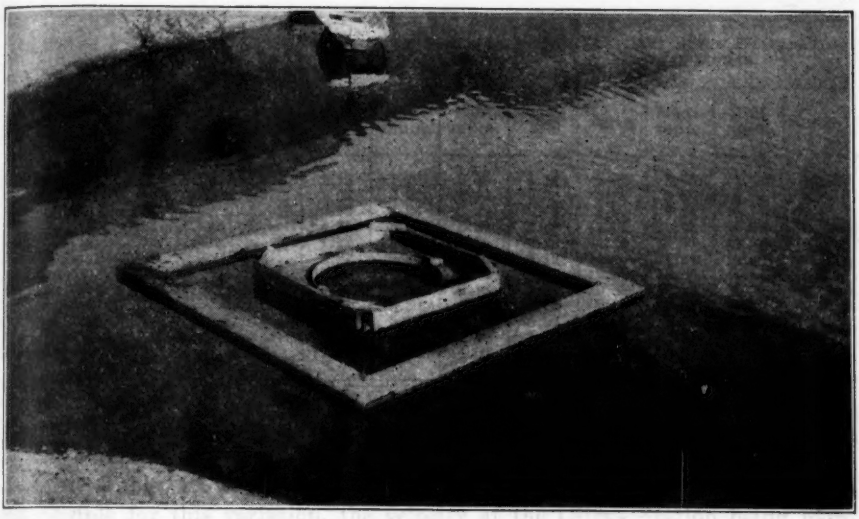

FIG. 2.—FLOATING EVAPORATION PAN AT THE EAST PARK RESERVOIR, ORLAND PROJECT.

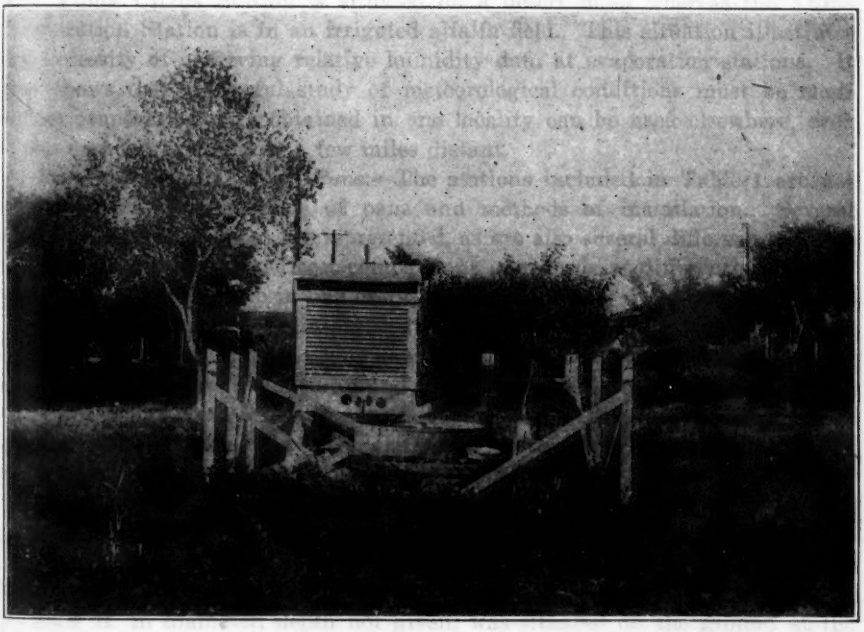

FIG. 3.—LAND PAN STATION AT MESILLA PARK, RIO GRANDE PROJECT, MAINTAINED BY THE NEW MEXICO AGRICULTURAL COLLEGE.

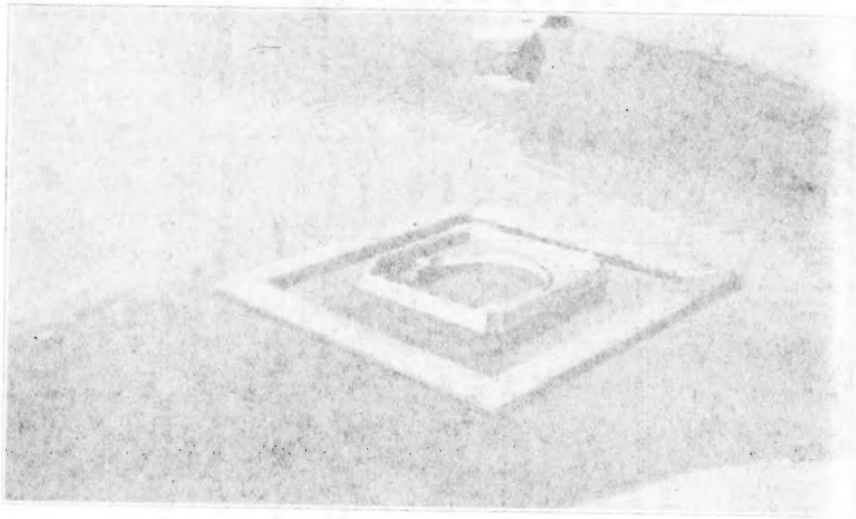

FIG. 2.—FLOATING EVAPORATION PAN AT THE SALT FARK RESERVOIR, ORLAND PROJECT.

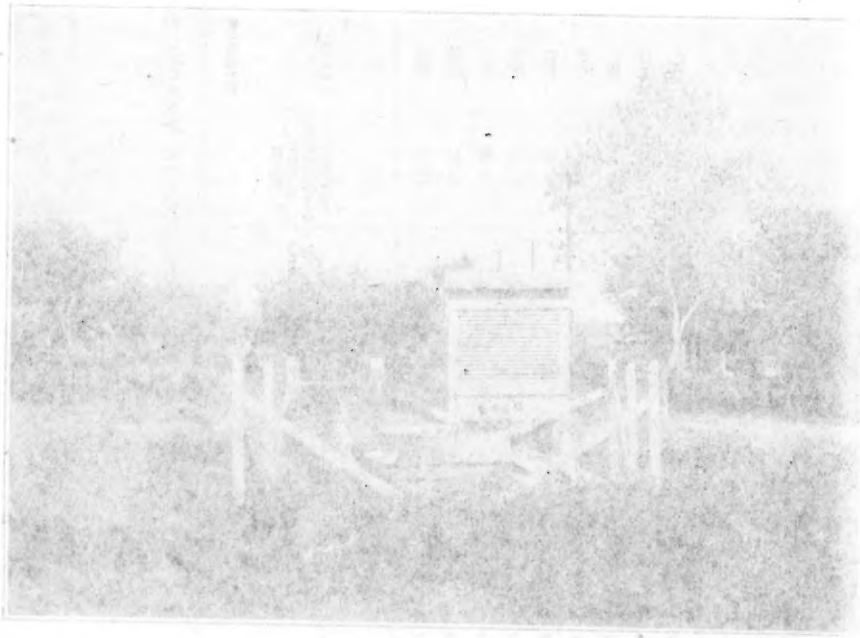

FIG. 3.—LAND PAN STATION AT MESILLA PARK, RIO GRANDE PROJECT, MAINTAINED BY THE NEW MEXICO AGRICULTURAL COLLEGE

TABLE 3.—RANGE IN MONTHLY EVAPORATION AT THE EAST PARK FLOATING-PAN STATION, AND THE ELEPHANT BUTTE LAND-PAN STATION.

Month.	EAST PARK EVAPORATION, IN INCHES.				ELEPHANT BUTTE EVAPORATION IN INCHES.			
	Mean.	Maximum.	Minimum.	Range.	Mean.	Maximum.	Minimum.	Range.
January	1.13	1.70	0.56	1.14	2.82	3.85	1.40	2.45
February	1.64	2.92	0.92	2.00	3.98	5.64	2.90	2.74
March	2.79	4.36	1.20	3.16	8.05	9.64	6.76	2.88
April	4.75	5.84	3.52	2.32	11.03	12.93	9.23	3.70
May	7.22	8.56	4.36	4.20	14.15	15.71	12.58	3.13
June	8.87	9.88	7.69	2.19	14.29	17.06	12.66	4.40
July	10.54	11.42	9.24	2.18	12.55	13.85	10.24	3.61
August	9.85	10.60	9.18	1.42	10.88	12.19	9.10	3.09
September	7.21	8.30	5.64	2.66	9.08	10.17	7.83	2.34
October	4.47	5.52	2.50	3.02	7.90	9.41	6.72	2.69
November	2.38	3.52	1.60	1.92	4.05	5.34	2.77	2.57
December	1.54	2.20	1.00	1.20	3.02	3.96	0.99	2.97

measured at the Yuma Citrus Station is about 70% more than that observed at the Yuma Evaporation Station. The diagrams in Fig. 4, platted from the data in Table 1, would indicate that differences in wind velocity were responsible for this variation, the velocity at the Citrus Station being more than twice as great as at the Evaporation Station. Although the greater wind velocity undoubtedly accounts for part of the increased evaporation, of greater importance is the much lower relative humidity at the Citrus Station. The Yuma Citrus Station is situated on a desert mesa whereas the Yuma Evaporation Station is in an irrigated alfalfa field. This situation illustrates the necessity of observing relative humidity data at evaporation stations. It also shows that a careful study of meteorological conditions must be made before evaporation data obtained in one locality can be used elsewhere, even if the new location is only a few miles distant.

Size and Installation of Pans.—The stations included in Table 1 are not consistent as regards kinds of pans and methods of installation. Several different sizes and shapes are represented, as are also several different styles of mounting. Fortunately, direct comparisons of records at differently designed installations are possible in one or two instances.

Two land pans were maintained at Jackson Lake Reservoir, in Wyoming, during the summer of 1924. One pan, 14 in. deep and 3 ft. square, was set in the ground with earth banked around the sides to the level of the water which was kept about 2 in. below the top. The other—a Class A pan, 10 in. deep and 4 ft. in diameter—was set on the ground with no earth placed around the sides. The evaporation from the former was only 72% of that measured at the latter, the monthly percentages varying from 69 to 75.

At Powell, Wyo., on the Shoshone Project, two land pans were maintained during the summer seasons from 1911 to 1919, inclusive. One pan, 8 in. deep and 3 ft. in diameter, was mounted on a tower 11 ft. above the ground. The other, 4 ft. in diameter, depth not given, was situated on the ground at the edge of an alfalfa field. The evaporation from the latter was only 62% that from the former. Individual monthly percentages varied from 50 to 84.

Floating and Land-Pan Evaporation.—Floating pans and land pans were maintained at the Nelson Reservoir, Milk River Project, Montana, from 1921 to 1923, inclusive. Both pans were the Standard U. S. Weather Bureau Class A type, 10 in. deep and 4 ft. in diameter. Pan No. 29 in Table 1 gives the mean records and the ratios of the floating-pan averages to the land-pan averages. The ratios vary from 0.60 to 0.76, averaging 0.68.

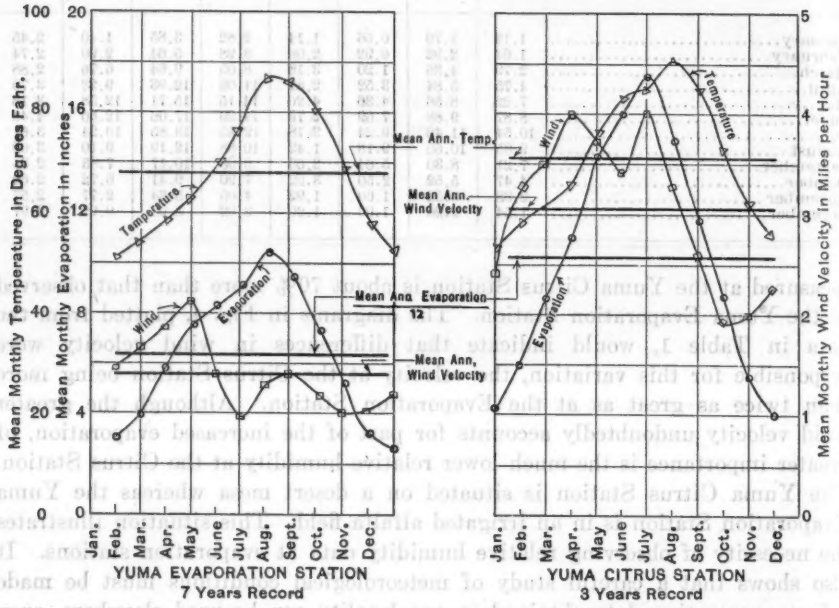
Mean Monthly Temperature in Degrees Fahr.
Mean Monthly Evaporation in Inches
Mean Monthly Wind Velocity in Miles per Hour
Temperature
Wind
Evaporation
Mean Ann. Temp.
Mean Ann. Wind Velocity
Mean Ann. Evaporation 12
Mean Ann. Wind Velocity
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.
YUMA EVAPORATION STATION
7 Years Record
YUMA CITRUS STATION
3 Years Record

FIG. 4.—COMPARISON OF RECORDS AT THE YUMA EVAPORATION STATIONS.

Similar comparative records were maintained at the Ralston Reservoir, Shoshone Project, Wyoming, from 1911 to 1914, inclusive, using circular pans 3 ft. in diameter, depths not known. The ratios of floating-pan averages to land-pan averages varied from 0.50 to 1.09, averaging 0.71. These values do not appear in Table 1 because the land-pan data given therein are for 1911 to 1916, inclusive.

Attempts to obtain similar comparisons at the Deer Flat and Arrowrock Stations on the Boise Project, Idaho, were made during the summer of 1916, but owing to sloshing troubles no very satisfactory information was secured. Sloshing troubles at the floating-pan station at Nelson Reservoir were partly eliminated by installing a No. 22 gauge, galvanized, sheet-iron honeycomb in the lower part of the pan, dividing it into compartments approximately 12 in. square and 6 in. deep.

Ice Evaporation.—Ice evaporation was measured at Saco, on the Milk River Project, Montana, during the winter of 1922 and 1923. G. E. Stratton, M. Am. Soc. C. E., Superintendent, reports as follows:

"An attempt was made to get some rough data on the winter evaporation. For this purpose, a pan, 14.85 in. in diameter (in which 1 oz. of water equals

0.01 in. in depth) and 20 in. in depth, was installed on a platform scale, at Saco. Some trouble was experienced with the pan bursting during thawing and freezing weather, but in general it was maintained during the winter months."

The results of Mr. Stratton's experiments are given in Table 4. It will be noticed that the average daily rates vary from 0.010 in., in January, when the mean temperature was 9.6° Fahr., to 0.070 in., in November, when the mean temperature was 31.1° Fahr.

TABLE 4.—ICE EVAPORATION ON THE MILK RIVER PROJECT, MONTANA, DURING THE WINTER OF 1922 AND 1923.

Month.	Number of days observed.	Total evaporation, in inches	Average Daily Rate.		Equivalent, in inches per month.	Mean* temperature, in degrees Fahrenheit.
			In inches.	In feet.		
1922:						
November	16	1.12	0.070	0.0059	2.10	31.1
December	13	0.17	0.013	0.0011	0.40	28.1
1923:						
January	20	0.20	0.010	0.0009	0.31	9.6
February	28	0.41	0.015	0.0012	0.41	9.8
March	10	0.56	0.056	0.0047	1.73	26.6

* For period of observations.

Practical Value of the Data.—Generally speaking, the mean data in Table 1 are not well adapted to theoretical studies of evaporation laws, or to testing the various formulas which have been prepared for calculating such losses. This is because of the varied types of equipment and installation represented, as well as of the interruptions in the records. Some of the longer, more continuous records at the warmer stations, such as those obtained at East Park, Elephant Butte, Roosevelt, and Yuma, may be suited to such purposes; although they are not as desirable as specially conducted experiments carried on by scientifically trained observers. However, it is believed that the data in Table 1 will be found of great practical value, not only to irrigation engineers, but also to those engaged on municipal water supply works, water power developments, or any other of the many engineering problems which require the use of evaporation records.

PIPE TUNNEL UNDER GOWANUS CANAL, BROOKLYN, NEW YORK

By L. S. Stiles,* M. Am. Soc. C. E.

To Be Presented March 3, 1926.

Synopsis

This paper describes the building of a tunnel under the Gowanus Canal, from the Citizens Works of The Brooklyn Union Gas Company, situated between Fifth and Huntington Streets, Brooklyn, N. Y. The purpose of the tunnel is to carry gas mains from this manufacturing plant and connect them to the distribution system, thus eliminating a lengthy and circuitous detour of pipe around the head of the canal.

The tunnel is 136 ft. from center to center of shafts, with an oval cross-section, 10 ft. 8 in. high and 16 ft. 1 in. wide. The tunnel and shafts are of steel plate, lined with reinforced concrete. It can accommodate two 30-in. and two 24-in. mains, with sufficient room for walkways on each side and ample space for repairs or replacement of the pipes.

The outstanding features in connection with this project, are as follows:

(1) The unique and inexpensive method used, which consisted of assembling the steel shell on ways, like a ship, launching it into a sheathed trench, and sinking it in position across the canal;
(2) The co-ordination of the many details of the work, obviating costly delays; and
(3) The perfect utility of the tunnel for the purpose for which it was designed.

The co-ordination of the work, insuring a minimum of delay, was doubly important by reason of the fact that all this construction had to be accomplished without interference with the extensive navigation in the canal. The Gowanus Canal is lined with large industrial enterprises, including a street railway power station, and their needs would brook no interference, however slight, in the navigation of the canal. The canal is closed to navigation from 5.00 P. M. to 7.00 A. M., and all day Sunday. Certain phases of the work had to be carefully co-ordinated with these periods.

Descriptive

The condition which prompted the construction of a pipe tunnel under the Gowanus Canal was the remarkable growth, within a few years, in the

Note.—This paper is issued before the date set for presentation and discussion. Correspondence is invited and may be sent by mail to the Secretary. Discussion on the paper will be closed in June, 1926, and, when finally closed, the paper, with discussion in full, will be published in *Transactions*.

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generating capacity of the Citizens Works of The Brooklyn Union Gas Company. One of the prime necessities for a gas plant is an abundantly adequate transmission system which will insure an easy "get-away" for the gas. The gas mains going through this tunnel provide a short cut eastward to the general transmission system of the Company at 12th Street and Second Avenue.

Prior to 1920, the capacity of this plant was 10 000 000 cu. ft. per day. An extensive construction program was launched about that time, and, in 1923, the capacity of the plant had been increased, by steps, to 29 000 000 cu. ft. daily, and additional transmission mains became absolutely necessary.

The Citizens Works is situated across the canal from some of its important mains (see Fig. 1). The original 24-in. main from the Citizens Works took a tortuous route in order to get around the head of the Gowanus Canal at Butler Street, and back to a 42-in. main that begins at the 12th Street Pumping Station, below Second Avenue. The length of the 24-in. main from the Citizens Works to the 42-in. main at Second Avenue and 12th Street is approximately 11 000 ft. Obviously, a much shorter line would be across the canal, and along Second Avenue to 12th Street, thereby eliminating approximately 8 000 ft. of transmission line.

As a matter of fact, the original route could not have been paralleled exactly with a second main as congested street conditions would have made it unreasonably costly; nor was it possible to go down Fourth or Fifth Avenues at a reasonable cost, because of the Brooklyn-Manhattan Transit Subway under Fourth Avenue and the elevated and trolley systems on Fifth Avenue. The route would necessarily have been to Sixth Avenue, and southward along Sixth Avenue, requiring an increased length of 2 200 ft. more, or about 13 000 ft. in all; and even this would have presented difficulties because of a large city water main running along part of the route in Sixth Avenue. In any case, the laying of a main along this route would have been complicated by the fact that it would cross the large Bond Street sewer, the Brooklyn-Manhattan Transit Subway line under Fourth Avenue, and the same Company's elevated structure along Fifth Avenue. A careful weighing of the relative cost of these two alternatives, both as to the initial cost of construction and as to the expense for loss of gas pressure which would have been entailed in the longer route, led to the conclusion that the line under the canal was the more advantageous in every respect.

As early as 1912, the accumulation of data relative to this work was begun. Consideration was given to an aerial bridge, to support an overhead pipe across the canal, but it was apparent that the lofty rigging of the boats using this canal would make this prohibitive, and that it would not be a desirable form of construction, subject, as it would be, to storms, etc. This overhead line was never seriously considered, but it was desired to exhaust the subject in deciding what ultimately should be done.

Having decided on a tunnel under the canal, the next questions were the method of construction, and the size and cross-section to be adopted. Consideration was given to the use of a shield, but such a method, with its attendant compressor plant and equipment, would have made the unit cost very high

for a tunnel of such short length. It was decided, therefore, to use the open-cut method, assembling the steel tunnel on shipways, conveniently spaced, and launching it into a coffer-dam purposely located, dredging across the canal, sinking the tunnel in place below the bed of the canal, and back-filling.

One of the factors influencing the choice of this type was the volume of shipping on the canal and the shortness of time available for work in it, limited to the hours of 5:00 P. M. to 7:00 A. M., and all day on Sundays, during which periods the waterway was closed to navigation. The extent of the shipping in this canal may be comprehended by the following figures, taken from U. S. War Department data, on the tonnage and value of shipping handled along this waterway:

"GOWANUS CREEK, N. Y.

	Tons.	Value.
1895	266 299	(Not known)
1896	220 748	(" ")
1897	215 352	$2 631 752
1899	237 300	3 048 390
1902	237 300	3 048 390
1903	237 300	3 048 390
1915	658 000	354 000
1919	1 607 261	18 802 541
1920	1 550 732	21 248 128
1921	2 839 689	96 215 400"

As to the size and cross-section to be used, a number of shapes and sizes were considered, and the final cross-section selected was of an oval form with parallel sides and semi-circular ends. This shape gave a maximum area with a minimum height, thus reducing the extent of dredging, and lent itself admirably to the reception of the four mains required, with walkways on each side and ample room for inspection, caulking pipe joints, or replacing lengths of pipe.

CONSTRUCTION

The merest outline of the steps in the construction will visualize the work as a whole before its details and interesting phases are discussed.

Property was acquired on the canal just opposite the Citizens Works and shipways were erected for the assembling and launching of the structure, which was delivered in sections. After launching into the coffer-dam previously built in the acquired property, the steelwork and concrete lining were partly completed. Meanwhile, the dredging for the shafts and the sheathed trench across the canal were completed. Finally, the steel tunnel, having been finished to a sufficient extent, was floated into place and sunk to its final position.

In Fig. 2 is shown a general view of the site in December, 1923. In the foreground the steel shell was erected and launched. This had formerly been a manure dump, and the clearing of the site for the coffer-dam and the shipways included a vast amount of fumigating for sanitary reasons. The total excavation for the shafts and tunnel section amounted to about 7 000 cu. yd.

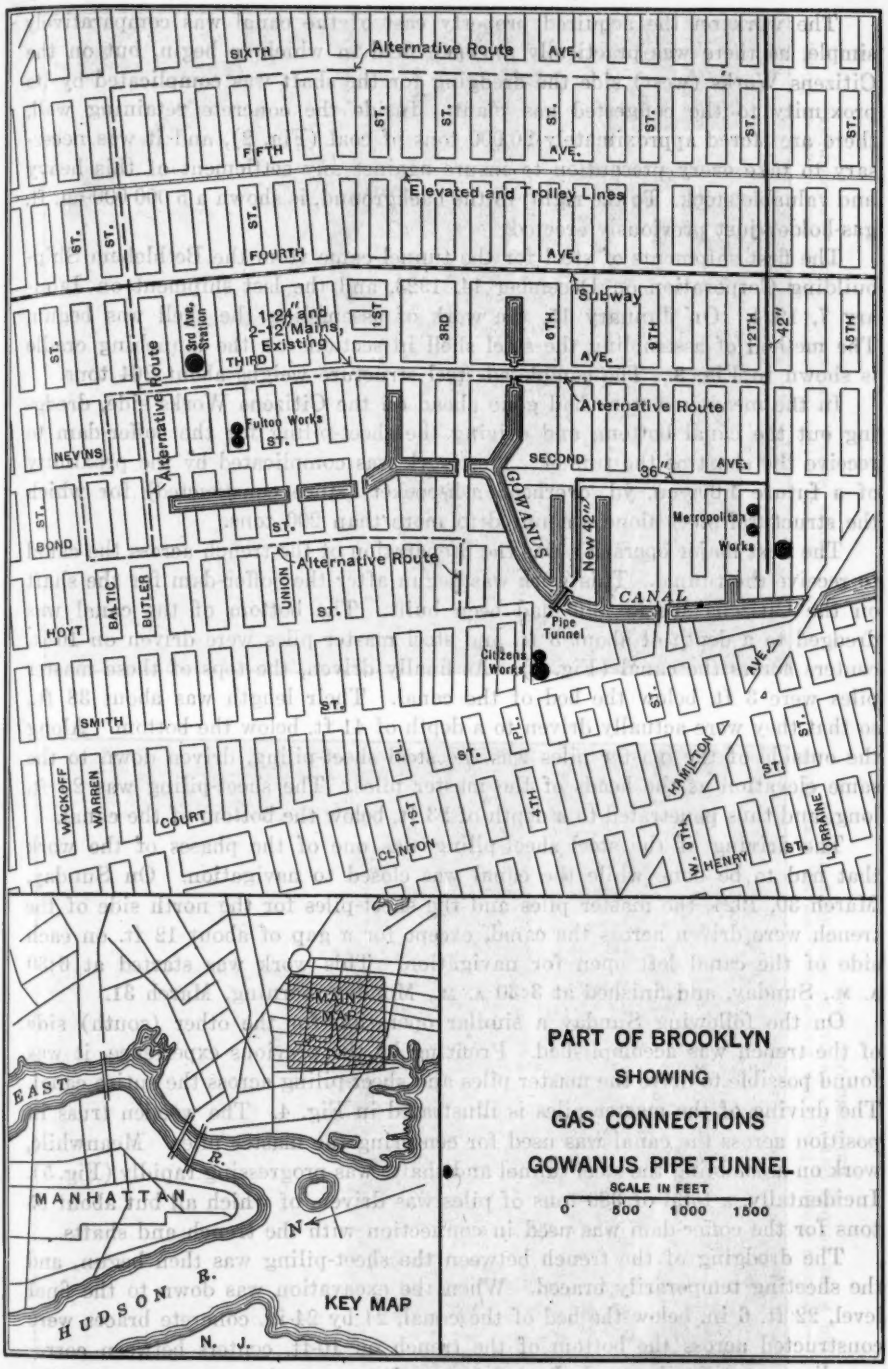

SIXTH
Alternative Route
AVE.
FIFTH
ST.
AVE.
Elevated and Trolley Lines
FOURTH
Subway
1-24" and 2-12" Mains, Existing
3rd Ave. Station
THIRD
1ST
3RD
6TH
9TH
12TH
15TH
42"
Alternative Route
Fulton Works
NEVINS
SECOND
36"
GOWANUS
New 42"
Metropolitan Works
BOND
UNION
Alternative Route
CANAL
Pipe Tunnel
HOYT
BALTIC
BUTLER
Citizens Works
SMITH
WYCKOFF
WARREN
COURT
CLINTON
HAMILTON
W. 9TH
HENRY
LORRAINE
MAIN MAP
EAST R.
MANHATTAN
HUDSON R.
N. J.
KEY MAP
PART OF BROOKLYN
SHOWING
GAS CONNECTIONS
GOWANUS PIPE TUNNEL
SCALE IN FEET
0 500 1000 1500

FIG. 1.

The work on the acquired property east of the canal was comparatively simple, as there was practically an open field in which to begin, but on the Citizens Works (west) side the dredging for the shaft was complicated by its proximity to the congested gas plant. Inside the concrete retaining wall, there are stored approximately 20 000 tons of coal (Fig. 2), and it was necessary to take every precaution to insure against any settlement of this heavy and valuable stock. To the right, in the background, is shown a 5 000 000-cu. ft. gas-holder just previously erected.

The first shipments of steel for the tunnel came from the Bethlehem Shipbuilding Corporation on December 14, 1923, and the last shipment on January 7, 1924. On January 15, the work of assembling the shell was begun. The method of assembling the steel shell in sections on the launching cradle is shown in Fig. 3. The completed steel structure weighs about 264 tons.

In the meantime work had gone ahead on the Citizens Works side, dredging out the canal bottom, and driving the sheet-piling for the coffer-dam to receive the shaft of the tunnel. This work was complicated by the proximity of a future 1 000-cu. yd. overhead ash-pocket (since constructed) for which the structural steel alone amounted to more than 200 tons.

The next major operation was the preparation of the trench across the canal to receive the tunnel. This work was begun after the coffer-dam for the shaft on the Citizens' Works side had been built. The bottom of the canal was dredged to a depth of about 3 ft. and steel master piles were driven on 10-ft. centers across the canal (Fig. 4). As finally driven, the tops of these master piles were 3 ft. below the bed of the canal. Their length was about 38 ft., so that they were actually driven to a depth of 41 ft. below the bottom. Along the outside of the master piles was the steel sheet-piling, driven down to the same elevation as the heads of the master piles. The sheet-piling was 20 ft. long, and thus penetrated to a depth of 23 ft. below the bottom of the canal.

The driving of the steel sheet-piling was one of the phases of the work that had to be done while the canal was closed to navigation. On Sunday, March 30, 1924, the master piles and the sheet-piles for the north side of the trench were driven across the canal, except for a gap of about 12 ft. on each side of the canal left open for navigation. This work was started at 6:30 A. M., Sunday, and finished at 3:30 A. M., Monday morning, March 31.

On the following Sunday a similar operation for the other (south) side of the trench was accomplished. Profiting by the previous experience, it was found possible to drive the master piles and sheet-piling across the entire canal. The driving of the master piles is illustrated in Fig. 4. The wooden truss in position across the canal was used for centering the master piles. Meanwhile, work on assembling the steel tunnel and shafts was progressing rapidly (Fig. 5). Incidentally, a total of 380 tons of piles was driven, of which all but about 55 tons for the coffer-dam was used in connection with the trench and shafts.

The dredging of the trench between the sheet-piling was then begun, and the sheeting temporarily braced. When the excavation was down to the final level, 22 ft. 6 in. below the bed of the canal, 24 by 24-in. concrete braces were constructed across the bottom of the trench on 10-ft. centers between corresponding master piles and the temporary braces at the top removed. The

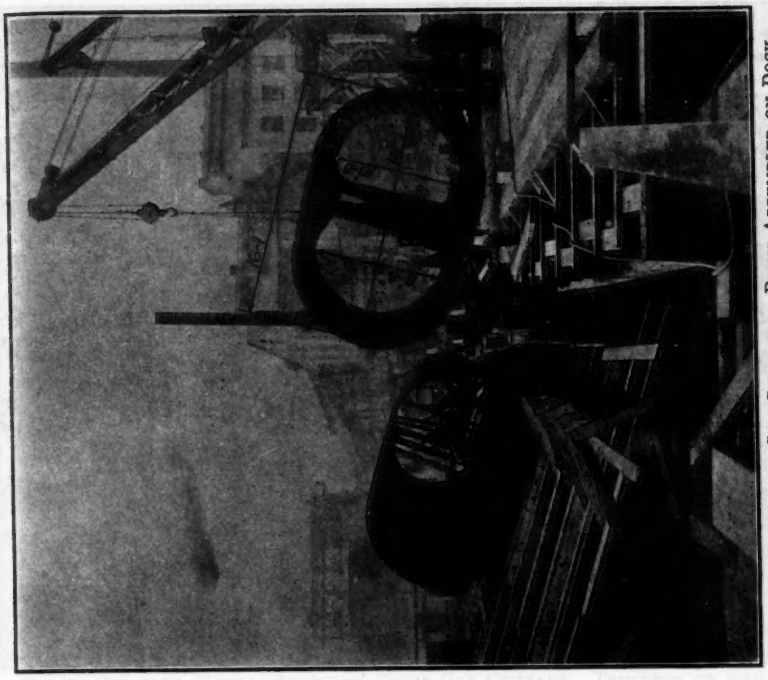


FIG. 3.—SECTION OF GAS CONDUCTOR BEING ASSEMBLED ON DOCK, CITIZENS WORKS, BROOKLYN UNION GAS COMPANY.

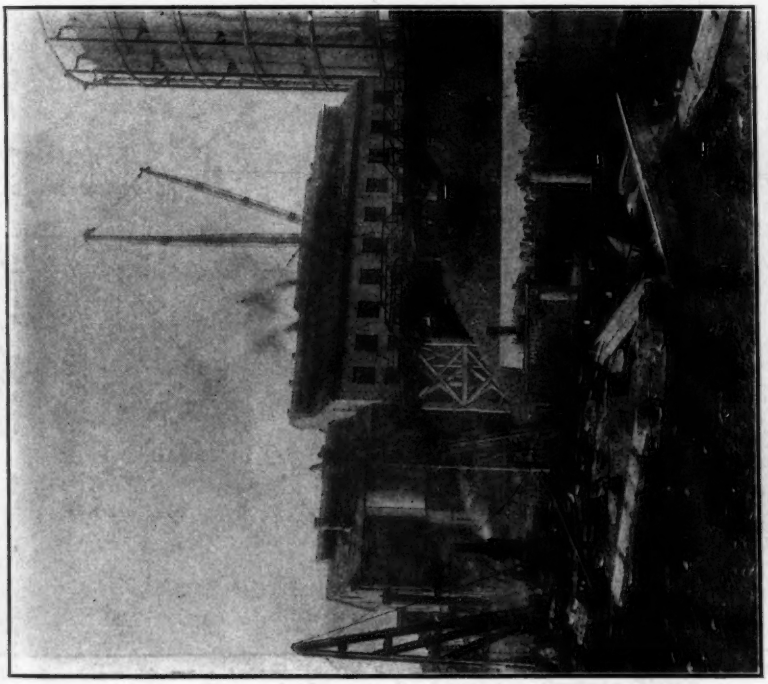


FIG. 2.—CITIZENS WORKS, BROOKLYN UNION GAS COMPANY, VIEW LOOKING WEST ACROSS GOWANUS CANAL, SHOWING PILE-DRIVERS REMOVING OLD DOCK.

FIG. 3.—CUTTING HOLE THROUGH CAISSON ROOF PREPARATORY TO SINKING PIPE TUNNEL UNDER GOWANUS CANAL.

FIG. 4.—CUTTING HOLE THROUGH CAISSON ROOF PREPARATORY TO SINKING PIPE TUNNEL UNDER GOWANUS CANAL.

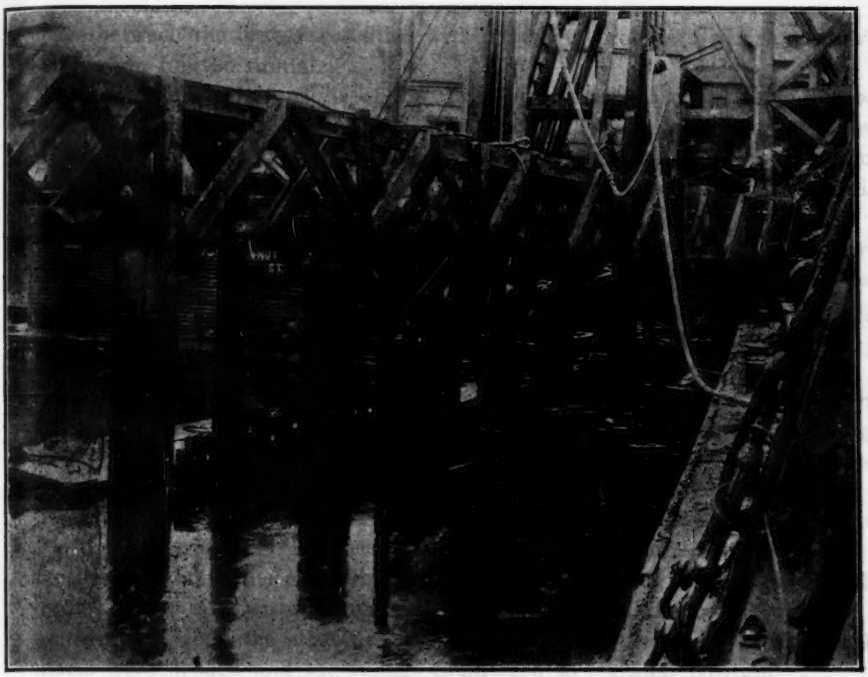


FIG. 4.—CITIZENS WORKS, BROOKLYN UNION GAS COMPANY. PROGRESS ON NEW GAS CONDUCTOR, APRIL 6, 1924.

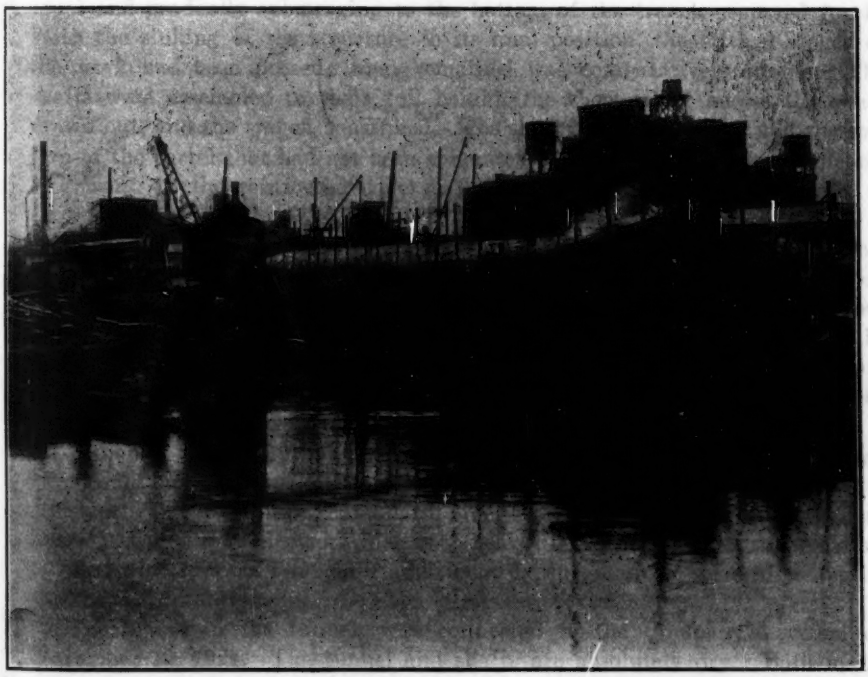
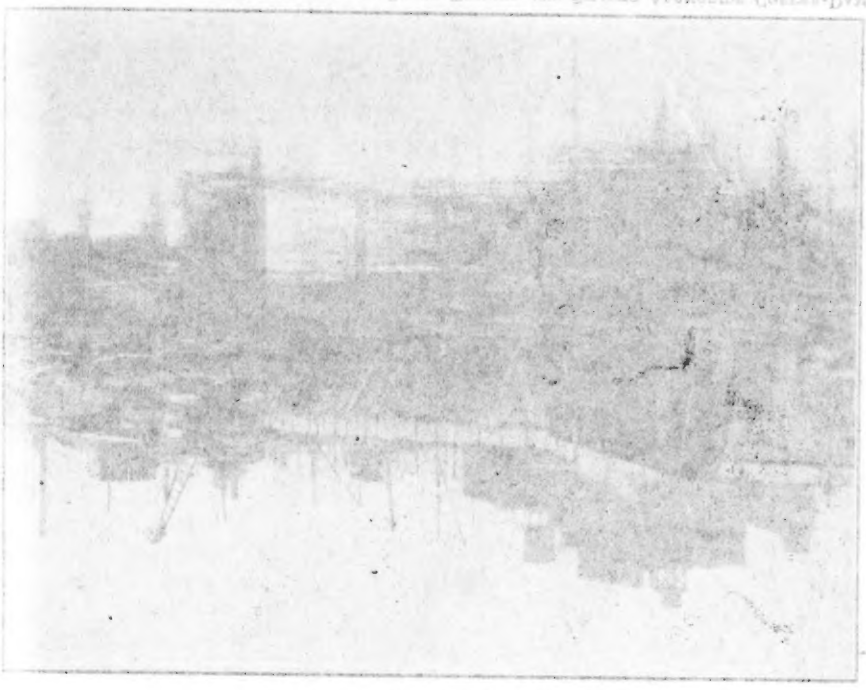


FIG. 5.—VIEW SHOWING ASSEMBLING OF STEEL TUNNEL AND SHAFTS ALONGSIDE COFFER-DAM INTO WHICH IT WAS TO BE LAUNCHED, CITIZENS WORKS, BROOKLYN UNION GAS COMPANY.



space between the braces was filled with gravel, to provide a secure and level foundation for the tunnel.

To meet the requirements of the War Department, it was ultimately necessary either to drive this sheeting down to 20 ft. 0 in. below mean low water, or to remove it entirely. The latter seemed the lesser evil, and all this piling eventually was pulled out, approximately 108 tons of sheet and master piles being recovered.

On April 19, everything was ready for launching the tunnel, the barrel and the two lower courses of the shafts having been assembled, riveted, and found absolutely tight. At 9:30 A. M., the structure was released from the ways and without the least difficulty floated into the coffer-dam prepared for it (Fig. 6).

While the tunnel was afloat in the coffer-dam, the reinforced concrete lining was placed in the barrel, and by Sunday, June 15, the concreting had progressed to a point where everything was ready to float the structure across the stream into position, and sink it. At this time the barrel was within a few inches of being awash. The great weight of the shafts, and, in counter relation, their greatly increased buoyancy per foot while sinking, made it necessary to brace the shaft shells preparatory to sinking. In order that the tunnel might sink on an even keel, the two ends of the barrel were bulkheaded, and when the structure had been towed to the correct position, the shafts were flooded and the tunnel submerged to its final position.

In Fig. 7 is shown the steel structure towed out to its position across the canal and gradually submerging to the bottom of the trench prepared for it. With the sinking of the structure to its final position, the critical stages of the work had been passed; what remained was comparatively simple. The shafts were assembled to their full height, the sheet-piling across the canal drawn out, and the trench back-filled. The shafts were concreted, and the section of the barrel that had not been concreted before the sinking, was finished. The work on the reinforced concrete lining in the barrel of the tunnel is shown in Fig. 8.

While all this work had been going on, the major part of the main from the east tunnel shaft to 12th Street and Second Avenue had been laid, and on the Citizens Works side, the line from the Pumping Station to the Citizens Works shaft had also been brought up to a point where it was ready to be connected to the mains going through the tunnel. Two 30-in. lines were laid through the tunnel, leaving ample room for two additional 24-in. lines in the future.

Although the tunnel had been built to stay perfectly dry as far as human skill could ensure this, provision was made for a possible flooding. Instead of American Gas Association standard pipe, American Water Works standard Class "D" pipe was selected, which is so heavy it will not float when the tunnel is filled with water. The slight additional expense entailed seemed to justify this extra precaution.

The mains in the tunnel were connected to the Company's system on November 13, 1924. The tunnel and shafts are absolutely dry. Both mains

were made tight under 10 lb. of air pressure before subjecting them to gas pressure, and as the working pressure of the gas is only one-half of that, the mains are perfectly tight. Fig. 9 shows the two mains laid through the tunnel.

Structural Steel Design

The tunnel is a combination of a rectangle 5 ft. 5 in. wide and two semi-circular sections of 5 ft. 3½ in. radii, making the width of the tunnel 16 ft. 0 in. inside the steel shell, and the height 10 ft. 7 in. To take up the pressure on the straight sections a longitudinal bracing truss was placed through the center of the tunnel as shown clearly in Figs. 8 and 9. This truss was not intended to act as a beam.

A number of pipe tunnels were investigated in the preliminary research. Fig. 10 shows some of the comparative cross-sections. The final selection is really a compromise between the other two, combining the advantages of both, without their drawbacks. This permits two 30-in. pipes on one side, and two 24-in. pipes on the other. The pipes shown at the extreme ends of the cross-section represent replacement sections and emphasize the adequate space available in which to work. As a matter of fact, this scheme was modified slightly. Instead of putting in two 30-in. pipes one on top of the other, they were placed side by side, and the two future 24-in. pipes will go over them; or, if it should be found necessary, two 30-in. mains can be added above, and the required clearances still maintained.

The shell is made up of ⅜-in. and ½-in. plates riveted to angles bent to the shape of the tunnel. The circumferential and tangential joints are covered with an exterior ½-in. butt-plate. All rivet holes were sub-punched and reamed, and all joints caulked.

The steel shell of the tunnel and shafts is lined with 12 in. of reinforced concrete. This lining is designed to support the entire load acting on the tunnel if, in the course of time, the outside steel should rust away. The longitudinal steel bracing through the center of the tunnel sets against the steel lining so that it will always function.

The plan and sections of the tunnel showing the general arrangement and the relation of the various appurtenances are indicated in Fig. 11.

Plotting Progress

The plotting of progress on plans and perspectives is an invaluable aid in co-ordinating the different phases of the work and in eliminating vexatious delays. It may appear elemental, but as a means of visualizing the work as a unit, it has no peer. It lessens the chance of halting the work for "lack of a tenpenny nail". There is a moment in almost every engineering project, when with the thousand loose ends that must dovetail not finished, or perhaps not begun, things look chaotic, and the engineer in charge, with all his planning and visualizing, trusts blindly to luck. At that moment the labor spent on keeping progress charts up to date more than repays its cost.

In looking over the actual work in its early stages the engineer is apt to be in the dilemma of the man who could not see the forest for the trees. A few plans and perspectives on which the progress of various items of the

FIG. 6.—CITIZENS WORKS, BROOKLYN UNION GAS COMPANY. PROGRESS ON NEW GAS CONDUCTOR, JUNE 2, 1924.

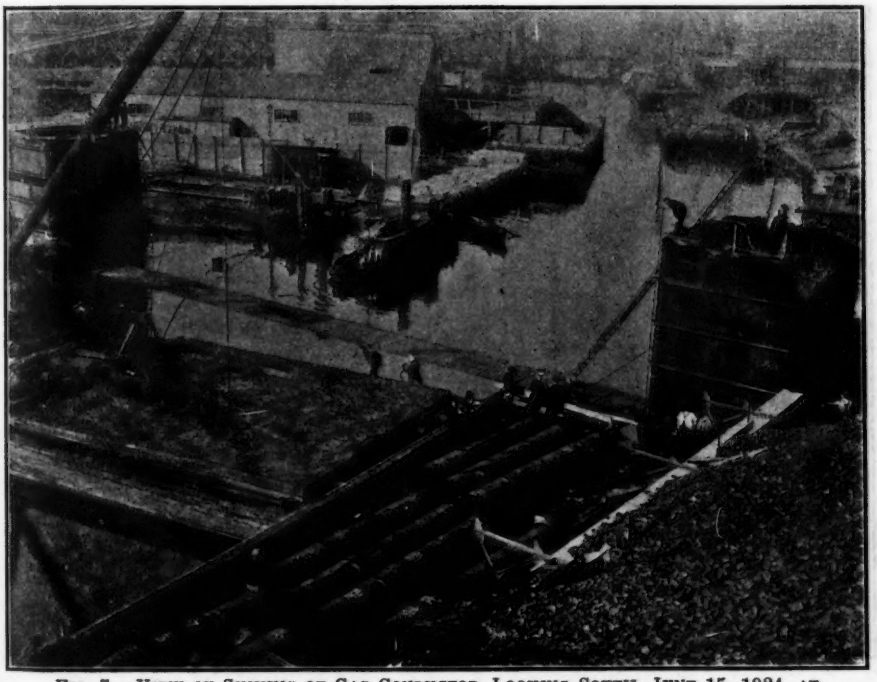
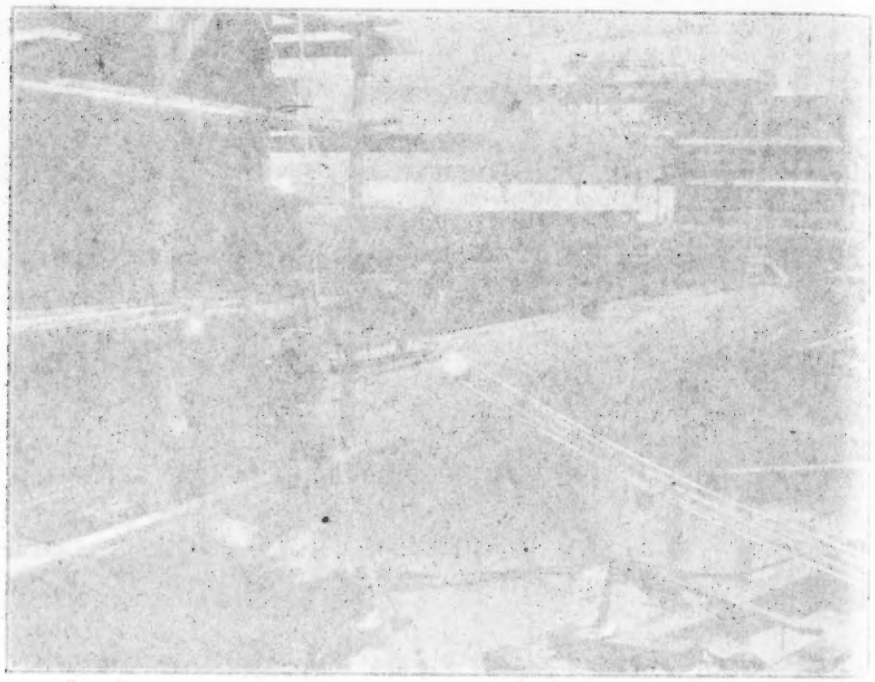
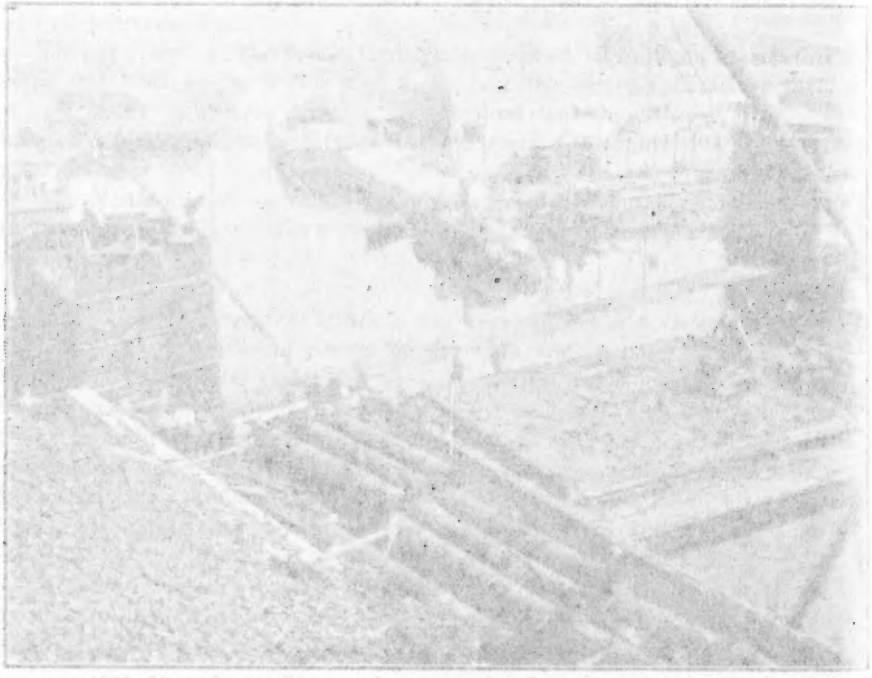


FIG. 7.—VIEW OF SINKING OF GAS CONDUCTOR, LOOKING SOUTH, JUNE 15, 1924, AT CITIZENS WORKS, BROOKLYN UNION GAS COMPANY.





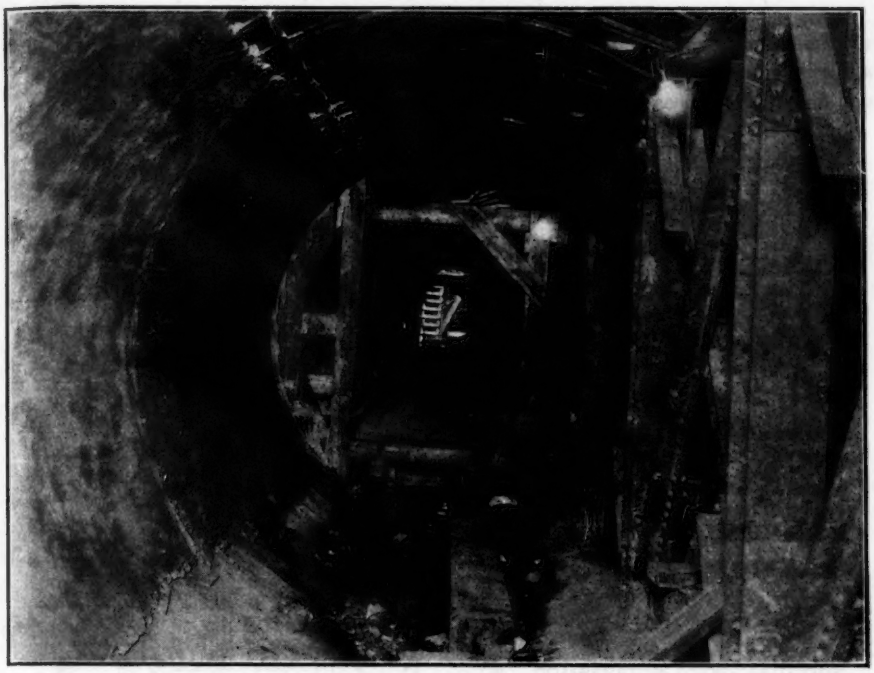


FIG. 8.—PROGRESS ON GAS CONDUCTOR, MAY 29, 1924, CITIZENS WORKS, BROOKLYN UNION GAS COMPANY.

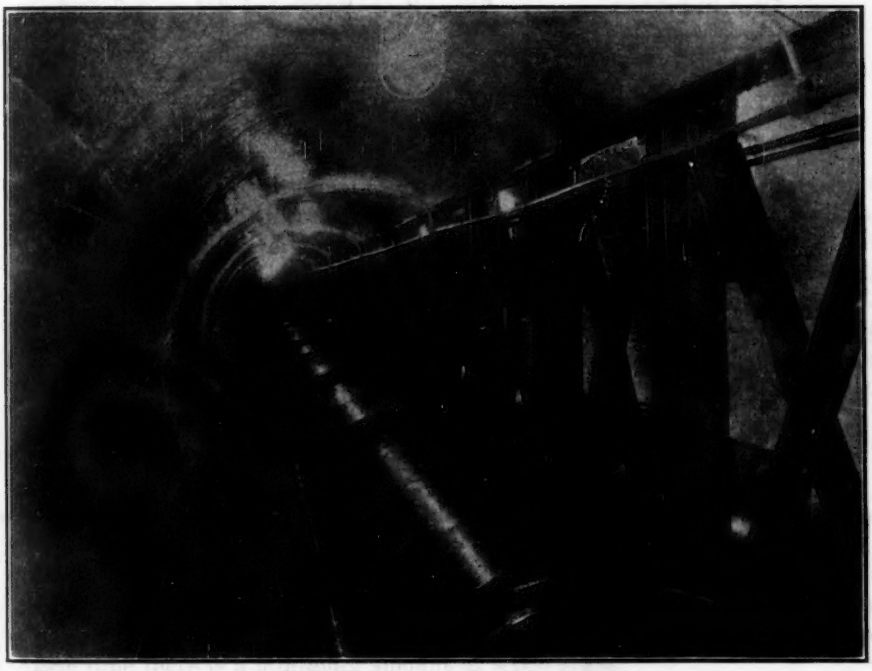
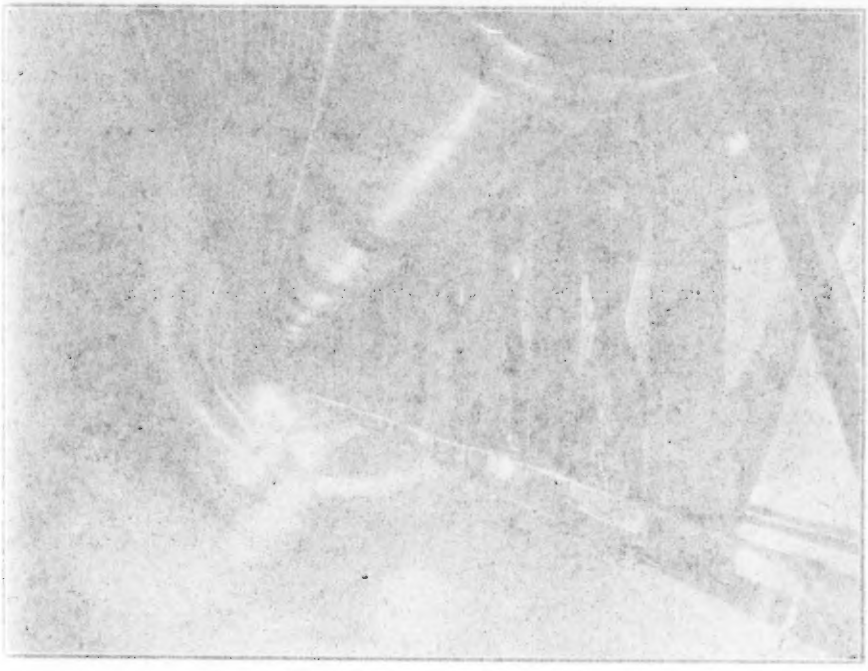
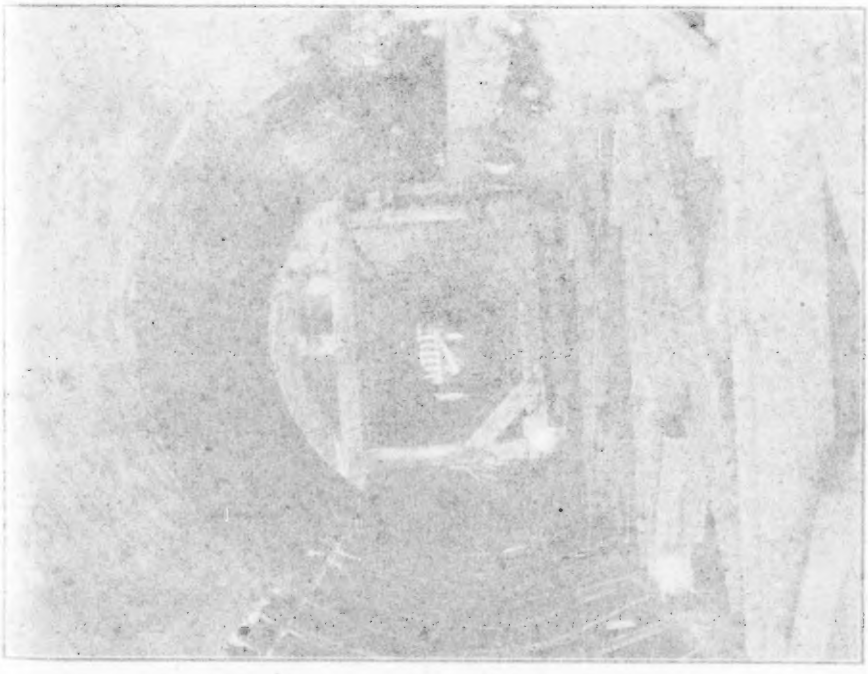


FIG. 9.—VIEW OF INTERIOR OF COMPLETED TUNNEL, CITIZENS WORKS, BROOKLYN UNION GAS COMPANY.





tunnel work were minutely and carefully plotted, will illustrate the methods used. During the progress of construction these charts hung on the walls of the field office, and a quick glance was all that was necessary to grasp the status of the work.

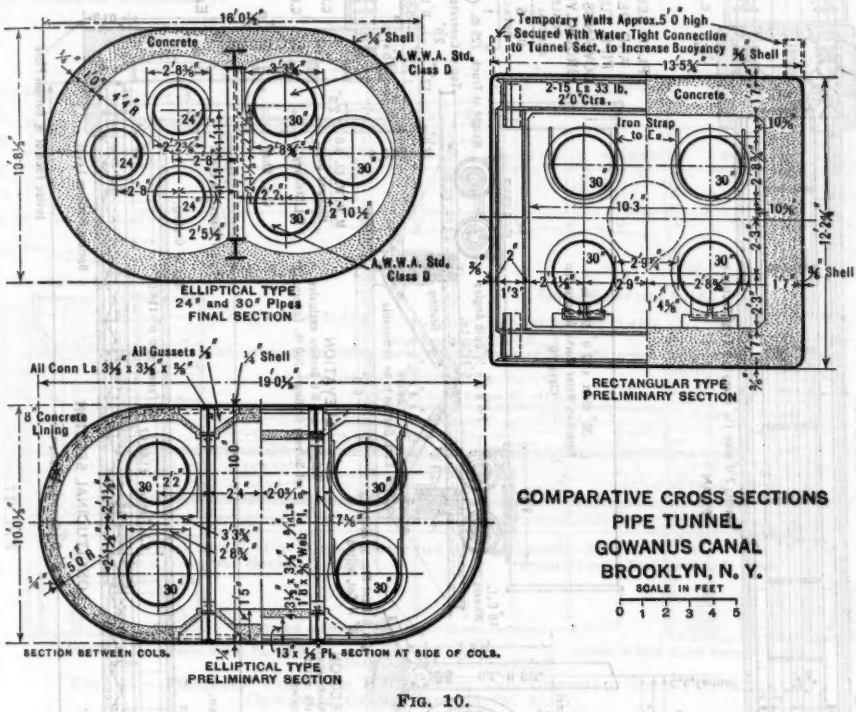


FIG. 10.

On a plan and longitudinal elevation (Fig. 12) was plotted a daily record of the dredging as it proceeded. Similar plans and longitudinal cross-sections were used for the delivery of the steel and for the progress of the riveting and of the concreting. On these plans a colored patch covering that part of the work which had been finished was filled in each day, so that there was available at all times a chronological record of the work and a comprehensive picture of what had been done, and what still remained to be done. Likewise Fig. 13 shows the percentage of riveting completed on the tunnel by days. It is self-explanatory.

GENERAL DETAILS

Each side of the tunnel was provided with a walkway and rails for a pipe truck to facilitate the handling of pipe (Fig. 11). The installation of additional 24-in. mains above the present pipes will require this equipment. It will be noted that there is ample room for handling spare pipe, for removing sections of the present main, and for caulking or repairing the pipes. At the same time there is a minimum amount of waste space.

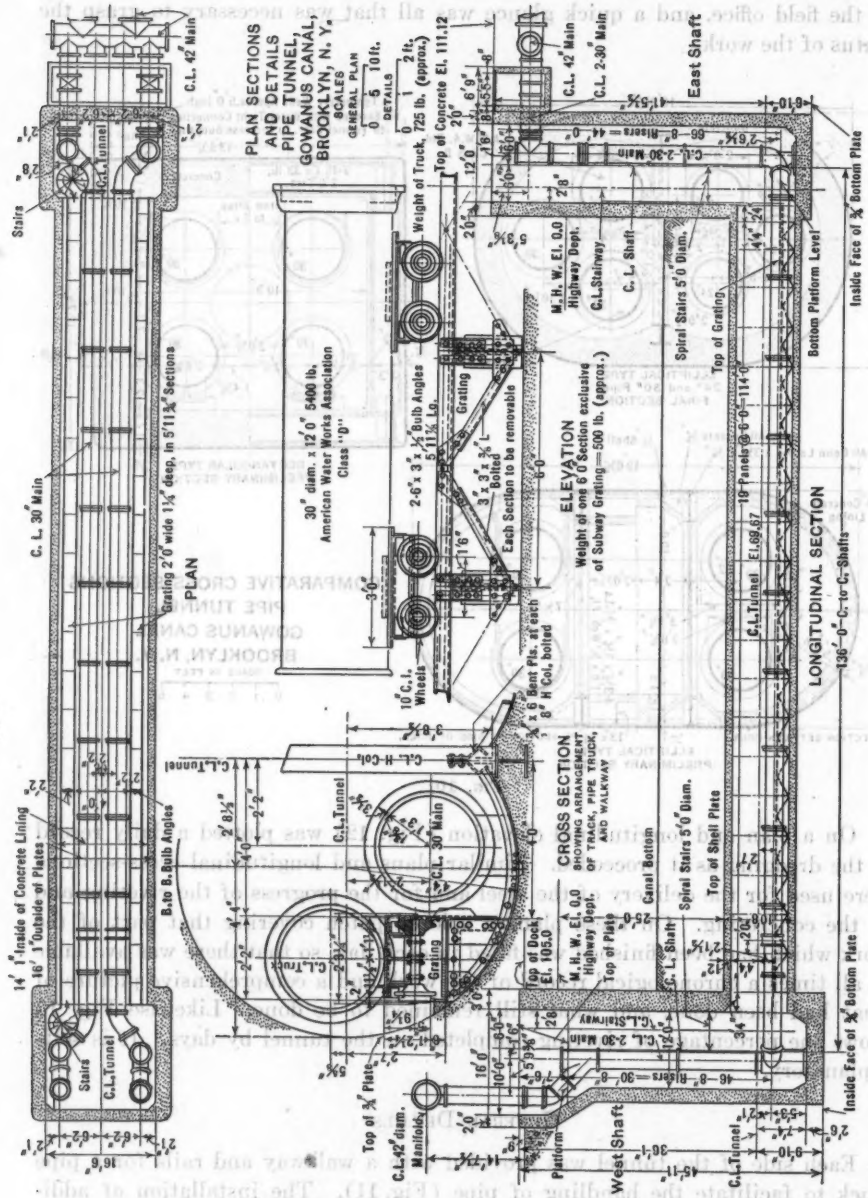

PLAN SECTIONS AND DETAILS PIPE TUNNEL GOWANUS CANAL, BROOKLYN, N. Y.
SCALES
GENERAL PLAN
DETAILS
PLAN
ELEVATION
CROSS SECTION
LONGITUDINAL SECTION
East Shaft
West Shaft
C.L. 42" Main
C.L. 30" Main
C.L. 2-30" Main
C.L. Tunnel
C.L. Shaft
C.L. Stairway
Stairs
Grating
Platform
Bottom Platform Level
Inside Face of ¾" Bottom Plate
Top of Grating
Top of Shell Plate
Canal Bottom
Top of Dock El. 105.0
M. H. W. El. 0.0 Highway Dept.
Spiral Stairs 5'0 Diam.
Weight of Truck, 725 lb. (approx.)
Top of Concrete El. 111.12
30" diam. x 12'0" 5400 lb. American Water Works Association Class "D"
Each Section to be removable
Weight of one 6'0 Section exclusive of Subway Grating = 500 lb. (approx.)
10" C. I. Wheels
8" x 6" Bent Pls. at each 8" H Col. bolted
Top of ¾" Plate
C.L. 42" diam. Manifold
C. L. 30" Main
Grating 2'0 wide 1¾" deep, in 5'11¾" Sections
14' 1" Inside of Concrete Lining
16' 1" Outside of Plates
B. to B. Bulb Angles
19 Panels @ 6'0 = 114'0
136'0" C. to C. Shafts

FIG. 11.

Provision was made for pumping out the "drip" resulting from the condensation of gas in the tunnel. The mains in the tunnel dip toward the Citizens Works side, which is deeper than the other, and contains the drip operated by a hand-pump from the platform level. No artificial ventilation was deemed necessary.

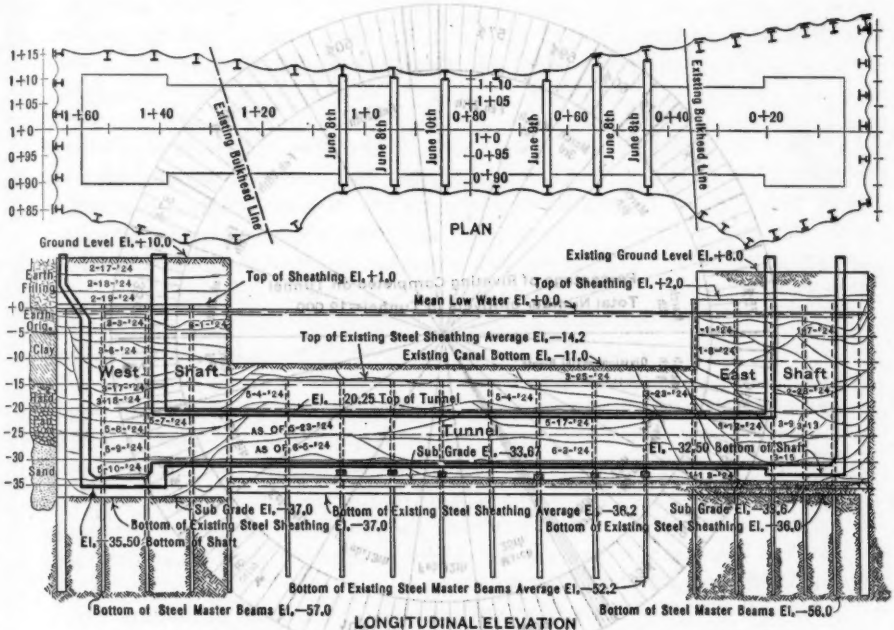

PLAN
Existing Bulkhead Line
Existing Bulkhead Line
Ground Level El.+10.0
Existing Ground Level El.+8.0
Top of Sheathing El.+1.0
Top of Sheathing El.+2.0
Mean Low Water El.+0.0
Top of Existing Steel Sheathing Average El.—14.2
Existing Canal Bottom El.—11.0
West Shaft
East Shaft
El. 20.25 Top of Tunnel
Tunnel
Sub Grade El.—33.07
El.—32.50 Bottom of Shaft
Sub Grade El.—37.0
Bottom of Existing Steel Sheathing El.—37.0
El.—35.50 Bottom of Shaft
Bottom of Existing Steel Sheathing Average El.—36.2
Sub Grade El.—33.6
Bottom of Existing Steel Sheathing El.—36.0
Bottom of Existing Steel Master Beams Average El.—52.2
Bottom of Steel Master Beams El.—57.0
Bottom of Steel Master Beams El.—56.0
LONGITUDINAL ELEVATION

FIG. 12.—PROGRESS PLAN FOR EXCAVATION AND DREDGING, PIPE TUNNEL, GOWANUS CANAL, BROOKLYN, N. Y.

The tunnel is lighted by electricity supplied from two separate and distinct services, one from the Citizens Works and the other from the Edison electric service of the 6th Street property of the Gas Company on the opposite side. At the entrance to the Citizens Works shaft, both feeders are connected to a three-pole double-throw switch, and a 4-circuit panel board was installed which is fed by either of the electric services.

The control of the lights throughout the tunnel and shafts is governed by four 3-way switches, two at the entrance to each shaft, fed by two circuits from the panel board. The lights are so wired that each set of 3-way switches controls every other light; thus, if a fuse should burn out on one circuit, the other circuit will continue to supply light. The same precaution has been taken with the Edison services; if one should fail the other is available, and either of them could be connected to the distribution panel boards of the double-throw switch. This 3-way switch makes it possible to turn the lights on or off at either entrance of both shafts. Galvanized hot dipped conduit is used. The fixtures are of the vapor-proof type, consisting of an outer glass globe protected by a wire guard.

In each fixture the screw-shell parts are connected to the grounded neutral wire of the lighting circuit. By this method, if, at any time, the screw shells should become loose and come in contact with the metal part of the fixture, since the fixture itself is grounded to the grounded neutral, no arc or short circuit would result.

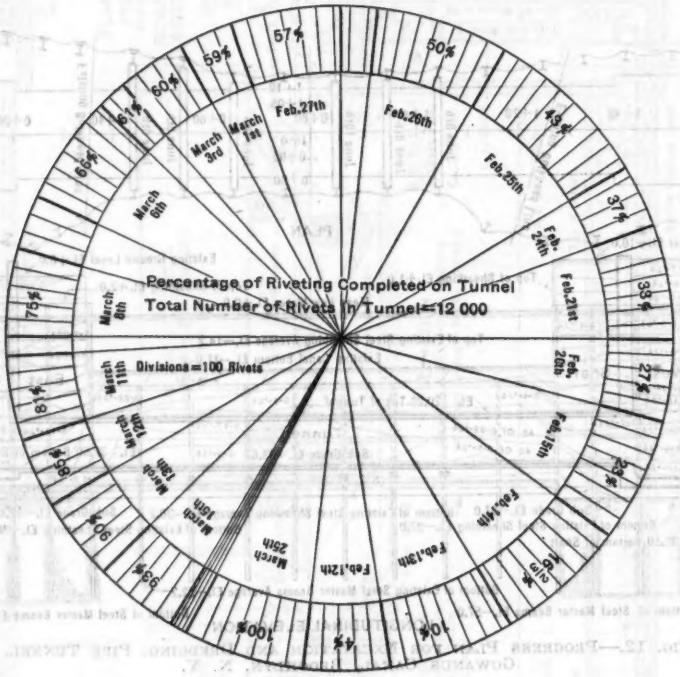


FIG. 13.

CONCLUSIONS

In the light of experience gained in the prosecution of the work, the following conclusions seem justified:

1.—That the method used in constructing this short tunnel was most economical, eliminating the use of an elaborate plant for driving the shield, and making it possible to use a cross-section best suited for the purpose; and

2.—That the oval section is preferable to a circular section and ideal for this particular work, allowing a minimum vertical height, with sufficient clearance above and between the pipes, and affording maximum utility for the purpose required.

ACKNOWLEDGMENTS

This work was executed under the direction of the writer by the Construction Department of The Brooklyn Union Gas Company. The writer acknowledges his indebtedness for the skillful aid of his assistant, Mr. W. M. Wepfer.

The steelwork was designed by Daniel E. Moran, M. Am. Soc. C. E., and the entire project was planned by Messrs. James C. Meem, M. Am. Soc. C. E., Chief Engineer of Frederick L. Cranford, Incorporated, Charles W. Staniford, M. Am. Soc. C. E., Mr. Moran, and the writer. Mr. Staniford was Consulting Engineer during construction. The Contractor was Frederick L. Cranford, Incorporated.

FIRE-BANKS FOR OIL STORAGE

Discussion*

By Messrs. Ralph J. Reed and H. H. Hall.†

Ralph J. Reed,‡ M. Am. Soc. C. E. (by letter).§—The paper describes a practical and constructive piece of research which is of great and immediate value not only to the petroleum industry, but also to safety, fire prevention, and insurance interests. The studies made and the conclusions drawn form the groundwork for the proper and economic design for permanent fire-wall protection of containers in which crude or fuel oil is stored. They are particularly useful in outlining means for modifying existing protective structures either permanently, or temporarily in the event of fire, so as greatly to increase their effectiveness against boil-overs.

Well constructed oil storage works develop few leaks that must be retained by secondary structures; the careful operation of tanks and reservoirs results in few spills; and, where proper principles of design are followed, as the author states, one may practically disregard the possibility of the rupture or collapse of a full container. It follows that fire-wall protection is primarily required as a measure to prevent the spread of burning oil if a boil-over occurs during a fire involving crude or fuel oils.

Although some fire prevention authorities have insisted on greater fire-wall capacity, it has been the general custom to assume that a fire-wall structure so built as to retain the entire tankage which it is designed to protect will usually be adequate to prevent the spread of burning oil in case of fire. The sole criterion has been the quantity of fluid which the protective structure will safely hold, without regard to the form of barrier or its relative location with respect to the containers surrounded. Where sufficient gound has been available, low earthen embankments with side slopes varying from 1 on 1 to 1 on 2 are common fire-walls. The usual field practice of the oil industry has been to build such earthen walls to heights of 5 or 6 ft., and to place them from 75 to 100 ft. from the tank enclosed. In other locations in which tanks must be spaced more closely, either on account of operating conditions or because of the restricted areas available, vertical walls of masonry or concrete are used. Mr. Hall's experiments, a short series of which it was the writer's privilege to witness, show clearly the ineffective barrier to the progress of a boil-over wave of high velocity furnished by the ordinary earthen embankment or even by a

* Discussion of the paper by H. H. Hall, M. Am. Soc. C. E., continued from September, 1925, *Proceedings*.

† Author's closure.

‡ Chf. Engr., Union Oil Co. of California, Los Angeles, Calif.

§ Received by the Secretary, October 26, 1925.

plain vertical wall of moderate height. Experience with earthen embankments during tank fires, together with observations of some recent fires in large reservoirs, serves to emphasize this fact strongly and to point to the value of some device for destroying the energy of the wave.

All the types of reinforcement for ordinary earthen embankments on which the tests were run are valuable from a practical standpoint, although the greater effectiveness of the low auxiliary wall placed quite close to the container, as compared with a coping of the same height placed on the top of the standard embankment, and its relatively shorter length, would seem to make it most desirable. Such walls may be built of concrete, tile, or brick. It is a question whether lighter material than, say, ⅜-in. steel plate, used as a coping, will successfully withstand the heat of burning oil, especially if repeated boil-overs occur; and in permanent installations even this material should be supported on posts, making a fairly expensive structure. Corrugated galvanized-iron sheets for temporary copings have been proven effective, but the writer questions their ability to resist the heat of successive boil-overs and is, therefore, of the opinion that their use should be confined to emergency installations as suggested by the author. Instead of steel plate, sheets of heavy asbestos board, properly supported, could be used.

The striking value of even a relatively small coping on a vertical wall appeared in the few experiments witnessed by the writer. No fire-wall of this type should be built without incorporating a coping in its design. Existing fire-walls can be readily and quite inexpensively modified, it is believed, either by casting a concrete coping in place, or by the expedient of attaching a metal plate to the top of the wall, as shown in two forms, (*A*) and (*B*), Fig. 20.

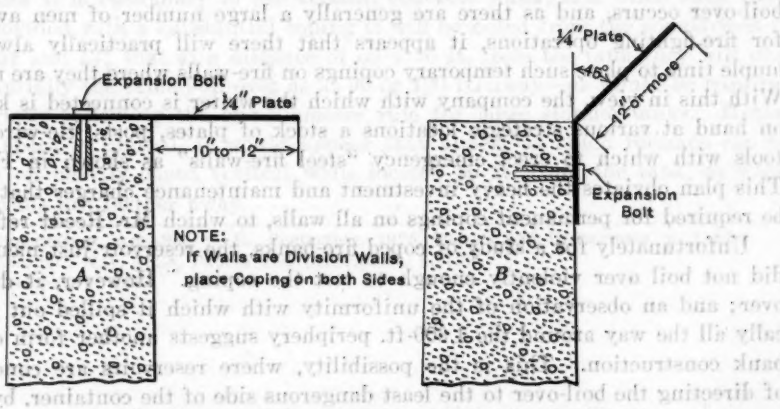


FIG. 20.—METAL COPING ATTACHED TO EXISTING FIRE-WALLS.

H. H. HALL,* M. AM. SOC. C. E. (by letter).†—The principles of fire-wall design developed in the paper are evidently proving of direct interest to the oil companies, several of which have carried the idea further and developed new methods for applying coping to their existing fire-walls. It is also interest-

* Chf. Engr., Standard Oil Co. (California), San Francisco, Calif.
† Received by the Secretary, November 7, 1925.

ing to note that in its recommendations to the National Fire Protection Association, the American Petroleum Institute has endorsed the coped type of wall.

Between January and November, 1925, the only oil fires in California of any magnitude have been in two large reservoirs; these were, incidentally, the only two reservoirs in the history of this industry on the Pacific Coast that have been fired by lightning. Although neither of these fires served to test the coped fire-wall idea completely, they both added to the knowledge of the general subject of fire-bank construction.

The first fire occurred at night in a reservoir about one-third full of heavy fuel oil. There was a substantial earth dike surrounding this structure, upon the top of which posts had been erected with the intention of establishing a permanent corrugated-iron coping somewhat similar to the metal coping on top of earth walls shown in Fig. 15.* The coping had been delivered to the work, but none of it had as yet been put into place. After the fire started about 150 men placed 4 000 ft. of this coping in about 2 hours. At this same fire, another gang of about 150 men, with the aid of three tractors and fifteen teams, built about 3 000 ft. of temporary earth wall with metal coping. For this work the metal and posts had already been prepared; but otherwise everything had to be done after the fire started.

These figures serve to illustrate the speed which can reasonably be expected under stress of fire conditions, and at night. In a recent daylight fire drill, a gang of 18 men erected 150 lin. ft. of steel-plate coping on top of existing fire-walls in 10 min.

As a crude oil tank or reservoir always burns a number of hours before a boil-over occurs, and as there are generally a large number of men available for fire-fighting operations, it appears that there will practically always be ample time to place such temporary copings on fire-walls where they are needed. With this in view, the company with which the writer is connected is keeping on hand at various strategic locations a stock of plates, posts, tie-wires, and tools with which to erect emergency "steel fire-walls" as shown in Fig. 21. This plan obviates the heavy investment and maintenance charges that would be required for permanent copings on all walls, to which Mr. Reed† refers.

Unfortunately for a study of coped fire-banks, the reservoir just mentioned did not boil over violently enough to test the coping. However, it did boil over; and an observation of the uniformity with which it spilled out practically all the way around the 1 600-ft. periphery suggests another form of fire-bank construction. This is the possibility, where reservoirs are concerned, of directing the boil-over to the least dangerous side of the container, by making the bank of the reservoir itself higher on the dangerous side. One reservoir embodying this principle was built four years ago; and there are now several reservoirs with auxiliary fire-banks built on top of their regular banks along the more dangerous side.

The other recent reservoir fire occurred in an isolated storage where there was practically no danger to other property. There was a substantial earth

* *Proceedings*, Am. Soc. C. E., May, 1925, Papers and Discussions, p. 741.
† See p. 83.

dike ranging from 8 to 16 ft. high, located about 200 ft. from the toe of the reservoir slope. The ground sloped gently toward the dike at a rate of about 5 ft. per 100 ft. This reservoir contained light crude oil, and boiled over with considerable violence. As there were no structures below it, no effort was made to erect a temporary coping; and this boil-over climbed the earth dike with apparently little loss of energy—which evidence simply serves to confirm the test results and considerable previous experience with walls of this type.

Mr. Shoemaker's statement* that "to be fully effective, the fire-wall must be placed one diameter from the shell of the tank", is in line with recommendations made by the Committee of the American Petroleum Institute, of which

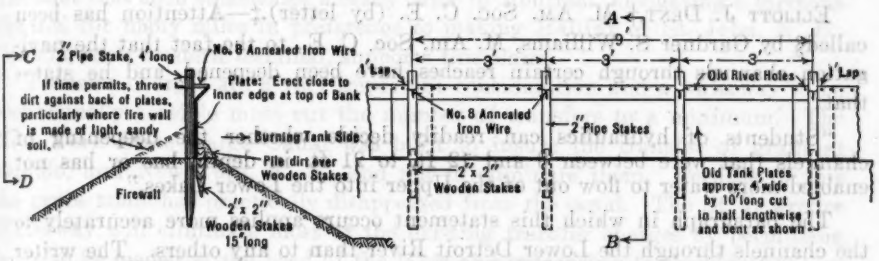

C
2" Pipe Stake, 4' long
No. 8 Annealed Iron Wire
If time permits, throw dirt against back of plates, particularly where fire wall is made of light, sandy soil.
Plate: Erect close to inner edge at top of Bank
1' Lap
Burning Tank Side
Pile dirt over Wooden Stakes
D
Firewall
2" x 2" Wooden Stakes 15' long
A
9'
3'
3'
3'
Old Rivet Holes
1' Lap
No. 8 Annealed Iron Wire
2" Pipe Stakes
2" x 2" Wooden Stakes
Old Tank Plates approx. 4' wide by 10' long cut in half lengthwise and bent as shown
B
SECTION A-B
ELEVATION C-D LOOKING TOWARD TANK

FIG. 21.—STEEL FIRE-WALL FOR REINFORCING FIRE-WALLS AROUND THAT PART OF A BURNING TANK OR RESERVOIR WHERE SPREAD OF BOIL-OVER WOULD BE ESPECIALLY DANGEROUS.

he was Chairman. This is the spacing which was considered ample to insure safety with all types of walls—particularly earthen dikes. In congested localities it is customary to use vertical concrete walls; and when such walls are provided with a substantial coping it will be noted from the comparison of Figs. 5† and 13‡ that at a distance of only 50 ft. from the tank they will stop substantially the same boil-over as at 90 or 100 ft. away—as would be required by strictly following the one diameter rule for large storage tanks. Hence, when space is limited, it would appear that almost as much reliance can be placed on heavily coped vertical walls located 50 ft. from the tank, as when such walls are farther away.

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* *Proceedings*, Am. Soc. C. E., September, 1925, Papers and Discussions, p. 1482.
† *Loc. cit.*, May, 1925, Papers and Discussions, p. 729.
‡ *Loc. cit.*, p. 738.

THE ST. LAWRENCE WATERWAY TO THE SEA

Discussion*

By Messrs. Elliott J. Dent, Kenneth L. DeBlois, and C. S. Riché.

Elliott J. Dent,† M. Am. Soc. C. E. (by letter).‡—Attention has been called§ by Gardner S. Williams, M. Am. Soc. C. E., to the fact that the navigation channels through certain reaches have been deepened, and he states that:

"Students of hydraulics can readily decide whether the deepening of channels that were between 8 and 12 ft. to 21 ft. in depth, has or has not enabled more water to flow out of the Upper into the Lower Lakes."

The paragraph in which this statement occurs applies more accurately to the channels through the Lower Detroit River than to any others. The writer is now in charge of this improvement and desires to lay a few additional facts before the members of the Society.

The aggregate width of all the natural channels through a section selected as more or less typical of the reach in question was approximately 11 000 ft. The improved channels are two in number, one being 600 ft. wide and the other 450 ft. wide, making the combined width of the deepened channels 1 050 ft., or about one-tenth the natural width. Part of the rock excavated from the improved channels has been carefully distributed in such a manner as to obstruct portions of the natural channels with a view to neutralizing the effect of the deepening. The question whether complete compensation has been effected is one that is still under consideration and measurements are made annually to supply the data for studying the subject.

The data so far accumulated fail to demonstrate conclusively any change in level at the Fort Wayne gauge about 10.5 miles above the head of the deepened channels. Any effect that may eventually be found at Fort Wayne will necessarily be considerably reduced before the lower end of Lake Huron is reached, as the intervening length of river channels is about 38.75 miles.

Kenneth L. DeBlois,‖ Jun. Am. Soc. C. E. (by letter).¶—There are several fundamental principles that should be considered before embarking on the improvement of any inland waterway.

Experience has proven that ocean steamships will go as far inland toward the center of production and industry as physical conditions will permit. As

* Discussion of the paper by Francis C. Shenehon, M. Am. Soc. C. E., continued from December, 1925, *Proceedings*.

† Lt.-Col., Corps of Engrs., U. S. A.; Dist. Engr., Detroit, Mich.

‡ Received by the Secretary, November 17, 1925.

§ *Proceedings*, Am. Soc. C. E., November, 1925, Papers and Discussions, p. 1873.

‖ Designer, with A. G. McGregor, Warren, Ariz.

¶ Received by the Secretary, November 21, 1925.

examples of ports developed at the head of ocean navigation may be cited: Philadelphia, Pa., 101 miles up the Delaware River; Baltimore, Md., 190 miles up the Chesapeake Bay; New Orleans, La., 100 miles up the Mississippi; Baton Rouge, La., 190 miles up the Mississippi; Portland, Ore., 113 miles up the Willamette River; and Montreal, Que., Canada, 1 000 miles up the St. Lawrence River.

Recognition of this fact in the past would have prevented uneconomical expenditures on such ports as Halifax, Nova Scotia, and Montauk Point, Long Island. The St. Lawrence Waterway fulfills this condition in that it will permit ocean steamships to go into a territory rich in industry and agriculture. The fact that such steamships already come to Montreal, navigating a tortuous channel for many miles in preference to making Halifax or St. John, N. B., Canada, their western terminal, supports this claim.

Transfer charges are a very important consideration and a waterway to be economically feasible must cut the number of transfers to a minimum. The New York State Barge Canal was built to insure the transfer of grain at Buffalo, and New York, N. Y., and not to eliminate them. Because of this the grain trade has practically disappeared from the canal. The St. Lawrence Waterway will eliminate most of the existing transfer charges by permitting boats to be loaded at the head of the lakes and proceeding to their destination without further transfer.

As a substitute for the St. Lawrence Waterway a deep canal across New York State has been suggested. Even if the immense sum of money were at hand to build a canal it is extremely doubtful whether ship owners would risk their vessels navigating a narrow channel for several hundred miles across the State. This project should not be seriously considered as a substitute. The proposed St. Lawrence Waterway must not be confused with a narrow ship canal. The author clearly outlines the use of dams and pools to give a wide channel for a greater part of the distance.

It has always puzzled the writer to understand why a certain group who are so adverse to the Great Lakes-St. Lawrence Waterway, where a large freight traffic already exists, at the same time advocate the expenditures of huge sums of money on barge canals where little or no traffic exists and where there is no prospect of any. On the editorial page of nearly every Middle Western newspaper will be found a platform advocating a barge canal as the one means of creating a permanent prosperity for their community. It is not the writer's intention to discuss these various projects but merely to emphasize that money should be spent on waterways which will be used when completed. The Great Lakes-St. Lawrence Waterway with ocean traffic at Montreal and the large existing traffic on the lakes can leave no doubt but that the proposed waterway will be used.

C. S. Riché,* M. Am. Soc. C. E. (by letter).†—Although the opening of the Great Lakes to ocean traffic seems inevitable, it has nevertheless been under discussion for a long time, and there seems to be enough opposition in certain quarters to keep it "under discussion" for many years longer.

* Col., U. S. A. (*Retired*), St. Louis, Mo.
† Received by the Secretary, November 27, 1925.

The situation is somewhat similar to that which existed 35 or 40 years ago, with respect to the Gulf of Mexico. For many years there was opposition to the creation of deep-water ports in the western part of the Gulf. The Southwest had then only recently been linked to other parts of the country by rail. Long rail hauls to Eastern ports were desired by many interests. Nevertheless the improvement of Galveston Harbor was actively undertaken in the early Nineties and was followed by the improvement of adjacent harbors at Sabine Pass, Port Arthur, Beaumont, Orange, Bolivar, Texas City, and Houston, Tex.

All these places are relatively near together. Their access to the ocean is either by Galveston Entrance or by Sabine Pass. They form the nearest ocean outlets for approximately one-fifth the area of Continental United States. Their commerce in and out, foreign and domestic, was very small before these harbors were deepened, and was subjected to lightering and other transfer charges which none but the most valuable freight could bear.

A record of this commerce has been kept by the United States Engineer Department, and appears in the Annual Report of the Chief of Engineers for 1924, the record for 1923 being the last one given.

Beginning in 1893, a few years before the first harbor was deepened, and listing the annual totals at intervals of five years discloses that the freight passing in and out of Galveston Entrance and Sabine Pass was as follows:

Year.	Short tons.
1893	415 403
1898	3 168 464
1903	4 867 965
1908	(Record incomplete)
1909	5 825 608
1913	9 915 829
1918	11 090 405
1923	24 622 522

The general development of the Southwest has had much to do with this great increase, but *vice versa* this development has chiefly resulted from the improvement of these harbors. There is no indication that the commerce of the Port of New York and other Eastern ports has suffered from this development but rather has profited and increased by reason of it.

The Great Lakes penetrate into the continent nearly as far to the westward as the Gulf of Mexico, into a section of greater natural resources than the Southwest. Although the North and Middle West have already been extensively developed, a much greater progress is possible and their accessibility to ocean traffic would stimulate this further development.

New commerce, increasing as has that of Galveston and Sabine Pass, could be expected at the Western Lake ports of Port Arthur, Ont., Canada; Duluth, Minn., and Chicago, Ill. Adjacent territory would develop more intensively, and Montreal, New York, and other Eastern ports, together with Canada and the United States as a whole would be benefited and not injured as a result.

Opposition to this improvement because of the fear of injuring existing business is ill-advised. Any development that would directly benefit so large a territory could not fail to benefit the continent as a whole. One such benefit not often mentioned would be the ability to build sea-going ships at lake shipyards. This can be done to-day but, with the exception of vessels of the small St. Lawrence Canal size, these lake-built boats have to be towed to the seacoast in sections and there assembled. Consequently, little such work is now done on the Great Lakes.

Steel is manufactured at a number of points on the lakes and there is an advantage in having shipyards close to the steel mills. With easy egress from the lakes for the product of the shipyards, it would be surprising if such an industry did not soon become extensive.

There would also be a benefit, as Mr. Shenehon points out, if lake vessels could be available for ocean service during the winter instead of lying idle for five months each year. The additional revenue thus obtainable might easily pay for such strengthening as would adapt these vessels to ocean service. At times, there are demands for additional carriers on the ocean, which would make such service quite profitable.

It is unnecessary to go back further than the dark days of the World War, when the submarine menace was at its height, to recall the demand for vessels for ocean service. Old boats, many far less seaworthy than the average lake vessel, were pressed into service and helped to save the day. Meanwhile, for five months each year, the great fleet of lake vessels lay idle, incapable of getting to the ocean. Had it been possible to transfer this fleet or any large part of it to the sea at that time it seems probable that the resulting saving would have paid the cost of improving the St. Lawrence River and the Great Lakes several times over.

It is to be hoped that these conditions will not recur, but they may, and it would seem advisable next time to be ready for them. Where that readiness will also result in a large commercial benefit, it would seem wise not to keep the subject "under discussion" indefinitely, but to take early steps to bring about results.

MULTIPLE-ARCH DAM AT GEM LAKE ON RUSH CREEK, CALIFORNIA

Discussion*

By Messrs. Luigi Luiggi, Thaddeus Merriman, M. M. O'Shaughnessy, J. B. Lippincott, and Oren Reed.

Luigi Luiggi,† Hon. M. Am. Soc. C. E.—This is a most valuable paper. Greater information is to be obtained from structures in a critical condition or from disasters than from those that are physically successful. Engineers, like physicians and surgeons, to become experts must operate on bad cases.

The paper emphasizes three important factors in a successful work, that is, design, materials, and supervision during construction. Dams, even in the most favorable circumstances, are difficult structures to build; and often the design has been considered more as a calculation of stresses than as a construction problem. Indeed, the design is an exceedingly important factor, for a badly designed dam is destined sooner or later to fail; but it is not sufficient merely to design the dam well—it is necessary to choose the materials best adapted to the conditions of the locality of the work. In particular, the cement must be of the highest class and be used in quantities substantially in excess of the theoretical proportions to allow for losses and deterioration.

It is easy to test cement in a laboratory and from the test to specify the exact proportion; but in the field, at a distance from the cement factory and lacking good material, it is necessary to adopt precautions that only practice can suggest. Consider the case of the Gem Lake Dam. The cement traveled over a normal railway, a narrow-gauge railway, then across a lake, and finally up a mountain side. It thus became very expensive; and when a material is costly there is always the temptation to use it sparingly. That is bad economy. Instead, more than the computed quantity should have been used to counterbalance the inevitable local losses: The wind blows some of it away, some of it sticks in the mixer and in the chutes. The best practice is to use 5 to 10% more than the theoretical quantity or even more than this under especially difficult conditions. It is bad economy to make weak concrete; this is dearly paid for later in the cost of repairs.

If the builders of the Gem Lake Dam had been more generous as to the cement, the concrete would have been less porous, and all the heavy expenditure to remedy this condition would have been avoided. As an adjunct to

* Discussion of the paper by Fred O. Dolson and Walter L. Huber, Members, Am. Soc. C. E., continued from December, 1925, *Proceedings*.

† Hon. Pres., State Council of Public Works; Insp. Gen. of Civ. Engrs.; Prof. of Hydr. Eng., Univ. of Rome; Member, International Technical Committee for the Suez Canal; Senator, Rome, Italy.

good design, a proper proportioning, a particularly good cement, and an excess of it, are indispensable for hydraulic work that must not be porous.

A third necessary precaution is a thoughtful and conscientious supervision of the work. To impress these three points more deeply on the minds of engineers—especially younger engineers—the speaker will instance a multiple-arch dam which, in height, design, and general dimensions, was practically a reproduction of the Gem Lake structure. The failure of this dam in the Valley of Gleno, in Italy, high up in the Alps of Bergamo, caused an immense loss of lives and tremendous damage. This dam was built about 6 000 ft. above sea level in a very cold climate and with difficult transportation.

As to design the Gleno Dam, although not excellent, was fairly good; but the materials were rather bad and the supervision even worse. The work was begun during the World War. Cement was scarce and costly because all building materials were going to the front. Workmen were scarce—either older than 50 or younger than 18 years of age—and unskilled. Supervision was almost completely lacking. The work was mainly under the supervision of a manufacturer, a self-made man, exceptionally clever as a cotton weaver, but with little experience in construction work. Under him were only ordinary workmen in charge of the construction. The work was carried out trusting to Providence.

During that time and for a few years after the war, Italy had a very inefficient Government; Socialists, not to say Communists, easily imposed on the Government officials, who were powerless. This employer was permitted, therefore, to proceed, carrying out this dangerous work without any technically competent man to advise him. Although Government officials occasionally went to inspect the progress and reported that the work should be stopped, should be partly demolished and rebuilt, still nothing was done to effectuate this. Further, because coal was extremely expensive and difficult to obtain and because hydro-electric power was the only remedy for the lack of coal, water was allowed to impound behind the dam, to produce as soon as possible the electric energy which, in turn, would keep the cotton mill busy and provide cloth for soldiers and work for the men at home. When the war was over, little was done to remedy this state of affairs.

As the dam was leaking in many places, some inefficient repairs were made, but these did not cure the consumptive condition of the whole structure. Then came a rather heavy storm, almost a cloudburst; the spillway was not large enough to discharge all the surplus water, which rose behind and nearly overtopped the dam. It was only a question of minutes, as described by an eye witness, before the dam began to crack and suddenly burst. This happened during the night. In the valley below several hundred people were killed and damage estimated at 150 000 000 lire was done. Many of the people were drowned in their beds.

As a result of an inquest into the cause and responsibility for this terrible disaster, some men were imprisoned and now have to answer for the disgraceful condition; but no one can resuscitate the drowned, nor compensate for the property destroyed. It is more to the point to search out the reasons for failure and thus avoid the mistakes made in the Gleno structure.

Although its design was questionable, with excellent materials and good workmanship, conscientiously controlled, the structure might have stood; but the defectiveness of the cement, the porosity of the concrete, and the badly built masonry which allowed water to percolate, were the main causes of the failure. In addition, there was an underlying mistake in the foundation, namely, a large mass of masonry which blocked up the bottom of the gorge and formed a platform on which the dam rested.

So poor was the cement mortar in this block, that it allowed water to pour out through the masonry. Worse still, it was not founded by means of proper steps cut out of the rock, but was laid on top of the natural surface which had been rubbed smooth by glacial action. No cut-off trench was provided to prevent leakage and consequent uplift. The only precautions taken were to grout iron bars into the rock where it was too smooth and thus, one might almost say, to nail the dam to the foundation. As a result of all these dangerous factors, the percolating water must have exerted an uplift, and when this occurred the masonry must have moved slightly, the iron bars lost their grip, and the whole structure slid down to destruction over a part of the dam where these treacherous foundations existed. Thus, the disaster was due primarily to lack of proper and skilled supervision.

State authorities should always insist on the advice and inspection of competent engineers. The author states that "for such localities [the Gem Lake Dam], a dam of the rock-fill type has many advantages over dams of other types." The speaker fully concurs in these conclusions; it is certain that in the case of the Gleno Dam, if a rock-fill structure had been built—which during the war could have been effected more easily than the multiple-arch dam—the disaster would have been avoided.

Regarding the use of preparations like "ironite", the speaker's experience of more than forty-six years convinces him that it is impossible to construct masonry and make it absolutely impermeable under a head of more than 40 or 50 ft. Not even a coating of neat cement, applied with a cement gun, will resist the percolation of water, because, besides stresses due to water pressure and change of temperature, there is another important effect, often disregarded, namely, the slight imperceptible movement of the crust of the earth to which the structure is subject almost daily. On a large scale these movements become earthquakes; but the very small daily tremors, noted only by seismography, cause the masonry to develop small hair cracks, which enlarge little by little and allow the water to percolate. In addition, the filling and lowering of the lake causes an elastic movement of the dam and of the foundation, which causes slight stresses that result in hair cracks. The great dam of the Murrumbidgee River in Australia is an illustration of this condition. The structure is about 240 ft. in height and the enormous pressure of the water causes actual movements in the foundation, as have been clearly recorded by seismographs set up in that locality.

Although it is possible to make masonry water-tight for some time by using special compounds, these infinitesimal movements, as years pass, will cause the masonry to crack. The speaker believes there is only one way of

making the surface impermeable—at least as nearly so as possible—and that is by covering the up-stream face with a membrane of sheets of bitumen, now commercially available in sizes about 4 ft. wide by 10 to 20 ft. long and ½ in. in thickness. These should be made to overlap and should be fused together with hot tools to form a plastic, continuous, and absolutely impermeable coating over the masonry. To protect this sheet from summer heat, floating logs, and blocks of ice, a revetment of masonry 1 to 3 ft. in thickness must be provided.

The durability of bitumen is attested by archeologists who have found in Nineveh sewers made with brick laid in bitumen which is still impermeable after 3 000 years. The speaker's belief that the Gem Lake Dam could have been properly protected in this manner, is strengthened by the instance of a rock-fill dam in the Alps, the Devero Dam. The up-stream face of this dam has a sheet bitumen coating protected by masonry 1½ ft. in thickness at the top, increasing to nearly 3 ft. at the bottom. After twenty-five years this dam shows no sign of leakage.

If any members of the Society have occasion to visit Italy on pleasure or on business, the speaker would feel privileged to assist them in the inspection and study of various engineering works and thus make their visit more profitable and pleasant.

THADDEUS MERRIMAN,* M. AM. SOC. C. E.—The buttresses and the arches of the Gem Lake Dam were both made of the same concrete. In the buttresses this concrete endured, while in the arches it failed. The paper seems to demonstrate that concrete which is serviceable in one exposure is not necessarily serviceable in another.

Mr. Flinn† has made a fair statement. Much good concrete has been placed, but there has also been some of doubtful quality. All concrete which disintegrates is not bad because the cement with which it was made was poor. Many of the troubles with concrete seem to be due to the fact that those who make it do not fully appreciate all the conditions which the process of concrete making involves. The speaker is free to admit, after many years of study, that this great problem still seems to be far from final solution.

On September 16, 1925, the speaker mixed a Portland cement with 43% by weight of water and poured a part of the mixture into one bottle and a second part into a second similar bottle. Forty-eight hours thereafter the first bottle was broken and the cement specimen so formed has since been exposed to the air. Twenty-one days later, on October 7, 1925, the second bottle was broken, and both specimens are here for examination. The first is soft and chalky, while the second is hard and dense. These specimens appear as if they had been made of entirely different materials, but, in fact, they are merely a demonstration of a difference in curing conditions. This is a simple experiment and one which all may profitably try.

The first of these specimens was treated in about the same way that most concrete is treated. It was put into a form, the form was pulled off at an early age, and Nature took its course. The specimen immediately dried out,

* Chf. Engr., Board of Water Supply, City of New York, New York, N. Y.

† *Proceedings*, Am. Soc. C. E., December, 1925, Papers and Discussions, p. 2015.

leaving the hydrations uncompleted. In the second specimen, however, the conditions were such that the hydrations went on to practical completion and the resulting texture indicates a strong, hard, and durable material. Concrete so cured would be a wonderful compound.

Between the texture and the quality of these two specimens there are many intermediate kinds of texture and quality. Under one condition of curing, one quality of product is realized; under other conditions, other qualities result. Some are good, some are indifferent, and some may be decidedly bad. So it is that one structure is very good, while another in the course of a few years may show serious disintegration. The quality of a concrete is not only a function of the conditions under which it was cured, but it is also dependent on the conditions of subsequent exposure. A thoroughly cured concrete, all other conditions being the same, will resist almost any exposure. A partly cured concrete, depending on the conditions of exposure, may or may not prove to be satisfactory.

The curing of concrete, and particularly that in thin sections, is, in large measure, dependent on the weather and moisture conditions during the first five or six days following its placement. If the humidity be low, nearly all the water necessary for the completion of the reactions will be lost, while, if the air be moist or water supplied, the hydration of the cement will proceed in an orderly manner. It has been claimed, if the process of hydration be interrupted, and if water be supplied at a later time, that the hydrations will resume themselves. No proof of this supposition has been adduced. Such evidence as the speaker has been able to develop is to the contrary. If the first specimen described is given plenty of water at any time after it has been dried out, it will never take on a texture resembling the second.

In the case of the Gem Lake Dam, all the conditions for rapid drying out were favorable. The high altitude, and the consequent low humidity, did its fatal work in withdrawing from those thin sections the moisture which was the very life blood of the cement. For weeks, and possibly months, that cement product was athirst. It was like the first specimen previously mentioned—spongy, chalky, and unstable. It could not resist the solvent effect of the water, and its constituents were leached out. On the other hand, that same imperfect cement product, in the buttresses, served its intended purpose. Freezing was not the primary cause of this disintegration. Frost merely hastened the visible evidences of deterioration. The quality of a cement product is always dependent on the maintenance of an environment in which the cement reactions may complete themselves. Good concrete can be realized only if it is kept for several days after making, under such conditions as will insure for the cement an opportunity to develop its latent possibilities. These conditions are clearly indicated by the bottle experiment which has been outlined.

In many cases the treatment accorded to concrete after placing is analogous to that of abandoning a two-day infant to its fate. On State highways it has been learned that curing is one of the most important single operations in the entire process of concrete making. In many other situations, however, concrete

after placing is allowed to shift for itself and then, as in a post-mortem investigation, the causes of disintegration are vainly sought.

The problem of durable concrete is the problem of durable cement. With a given cement the best results are not possible of attainment unless that cement be used with understanding skill of workmanship. Cements differ among themselves and results which can be obtained with one cement are impossible with another. The task which confronts us is twofold: First, we must learn to differentiate cements with respect to the quality of the hydration products they are capable of producing; and, second, we must learn how to use a cement of given quality so as to secure the results which any particular situation demands.

At present, none of these matters is susceptible of standardization. Standards of measure are useful as means of convenience and of comparison. Standards of quality and of methods operate to suppress individual initiative. Standards imply averages, and if quality of product be a consideration they tend toward mediocrity.

In this great matter of concrete and of cement we cannot rest content with averages and with mediocrity. Until the uttermost of the problem is within the grasp of our understanding we must do no less than seek for the very best within the present possibility of attainment.

M. M. O'SHAUGHNESSY,* M. AM. SOC. C. E. (by letter).†—The authors are to be commended for their frank discussion on the repair of the buttressed arch dam in the high Sierras near the State of Nevada, south of Lake Tahoe, California. No estimate has been furnished as to the resultant losses sustained by the Nevada-California Power Company due to the two years failure of this dam to function in storing water, which losses must have amounted to far in excess of the cost of initially building a properly designed dam of a type suited to the conditions and available materials. It is hoped they will furnish these data before closing the discussion, so that a frank statement of all conditions may be available to engineers interested in this type of dam.

The building of this dam followed the conventional design for this type of structure at the date of its construction (1916), and the failure is not primarily due to theoretical design features. Its designer, after describing it in a paper‡ before the Society, deservedly received the Norman Medal. He was retained by the writer in 1917 to aid Assistant Engineers McIntosh, Nishkian, and the late Mr. Wood in designing the Lake Eleanor Dam, 70 ft. high, 1 200 ft. long, on the Hetch Hetchy Project. Further studies on the design for this type of dam resulted in several beneficial modifications being incorporated into the Lake Eleanor Dam,§ which is a structural and economic success.

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* City Engr., San Francisco, Calif.

† Received by the Secretary, October 26, 1925.

‡ "Multiple-Arch Dams on Rush Creek, California," by L. R. Jorgenson, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 850.

§ *Engineering News-Record*, September 4, 1919, p. 455.

The true principle controlling dam design and construction was tersely elucidated by Mr. George L. Dillman,* namely, that there must be only one impervious surface next the water, and that this face must be adequately supported by solid masonry, buttresses, rock, or earth-fill, depending on the locality and the type of dam.

The term "multiple arch" as applied to this type of dam is a misnomer; there are so-called "multiple-arch" dams in India 120 years old, and the name is meaningless as to design. This particular type of dam survives because of the buttresses, and the principles and reason for its selection were first described (after a structure built in Oregon) in 1902 by Mr. Dillman,† and due credit should be given him for his initiative in recommending it.

Overtopping the Gem Lake Dam with 4 in. of water is not a very severe test. The Upper Otay Dam (a single arch), in San Diego County, 84 ft. high, built by laymen without engineering skill, was topped to a depth of 30 in. without injury. However, an enlarged spillway now prevents a recurrence of this happening.

The sections are so thin in the buttressed arch type of dam that it is essential to have the concrete of such density that it will be water-proof, and this can be accomplished only by a careful selection and proportioning of the materials, avoiding an excess of water in mixing, and thoroughly spading the concrete after it has been deposited in the forms.

The writer's impression of the value of alleged water-proofing compounds on concrete surfaces subjected to water pressure is in entire agreement with the experience of the authors; that is, these compounds have no worth except to their manufacturers and salesmen.

The concrete in the Gem Lake Dam failed because it was defective in that it was porous, thus allowing the water to penetrate it, and, with the heavy frosts and ice formation, permitting destructive disintegration to set in.

Why was the concrete porous? It may have been due to an excess of granite, mica, dust, and clay in the sand, or defective cement; or as Mr. Jewett states:‡

"* * * the presence in rock used as aggregate—which was obtained from material near the site of the work—[cheaply, of course, which feature should not be considered in dam construction] of thin, flat, smooth-surfaced pieces, which would have somewhat the same effect in concrete as mica in a sand mortar."

This class of rock prevents the proper interlocking of the remainder of the rock in the mass, and has a tendency to reduce the density of the concrete; and it is in such a situation that the experienced man, with judgment of materials, is needed fully as much as the designing engineer or the testing laboratory in determining the character and suitability of the materials to be used in the structure.

In building the 396 000-cu. yd. concrete dam at Hetch Hetchy, the engineers had quite a controversy with the contractors on their initial attempt to use a

* *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 52.

† "A Proposed New Type of Masonry Dam," by George L. Dillman, *Transactions*, Am. Soc. C. E., Vol. XLIX (1902), p. 94.

‡ *Proceedings*, Am. Soc. C. E., September, 1925, Papers and Discussions, p. 1455.

readily accessible, cheaply obtained sand. Exhaustive tests were made on samples of concrete and briquettes from this sand. They were found to be defective and failed to show the proper strength and hardness characteristics under time tests. The chemist's analysis subsequently disclosed the fact that the sands were heavily coated with a black compound which was largely tannic acid (from a dense grove of oak trees) and which caused the trouble.

Three miles up stream from the dam, at the junction of Tuolumne River and Rancheria Creek, there was found a bank (250 000 cu. yd.) of excellent sand which, after some protest on the contractor's part (because of the necessity of building a 3-mile narrow-gauge railroad to reach it), was used with very satisfactory results.

Evidently, Mr. Jewett's experience as a testing engineer indicates a doubt as to the value of the ordinary contract system for dam construction, since the tendency is to slight this class of work because of the contractor's desire for speed of operation and profit; this agrees in general with the writer's ideas. It may be mentioned in this connection that Lake Eleanor, Early Intake, Cherry Diversion, and Priest Dam (145 ft. high) were not built by contract, but in a very satisfactory manner under the direct charge of the resident engineers; and the O'Shaughnessy Dam at Hetch Hetchy was built by contractors who faithfully and promptly complied in every respect with the wishes of the resident construction engineer, who was assisted by a staff of twenty men, inspecting and testing.

In December, 1923, the Gleno Dam (buttressed arch type) failed in Italy, drowning 500 people and damaging property below to the extent of $5 000 000. No doubt, inadequate preparation of the foundations, together with defective lime masonry and concrete, put in without engineering control by the overzealous contractor in critical parts of the structure near the base, without Government supervision, caused the accident.

There is now a barrage of literary experts for contractors and damage claimants mailing reports back and forth on the causes of and responsibility for this failure, with one certain result, namely, that Premier Mussolini and the Italian Government are allowing no more questionable dams to be erected by any one, and have commissioned skilled Government engineers to inspect all materials and supervise all designs, so that there will be no repetition of the Gleno Dam disaster.

It is a pleasure to note that an engineer of extensive construction experience, such as Mr. Huber, endorses the virtues of a rock-fill dam for this type of location, with its abundance of cheap rock, thereby saving the expensive hauling of cement.

The Morena rock-fill dam, built under the writer's direction, survived the cloudbursts of 1916 in San Diego County, California, when the Lower Otay Dam failed, and when every county and railroad bridge in that community was destroyed by floods.

The idea of the rock-fill type of dam is traveling eastward, crossing the Mississippi River, as witness the Dix River Dam, 280 ft. high, in a limestone formation, in Kentucky, built for the Middle West Utilities Company by L. F. Harza, M. Am. Soc. C. E., to conserve water for power.

Although time and the financial exigency in 1917-18 compelled the City of San Francisco to build the buttressed arch Eleanor Dam (forming an 8 000 000 000-gal. reservoir at a cost of less than $300 000), definite plans are now being made for practically scrapping this structure, and substituting the ultimate dam of the rock-fill type at this point. This rock-fill dam will be below the present dam, using the 70 ft. concrete buttressed arch for the lower part of the up-stream toe or water-tight skin, and carefully placing and thoroughly grouting rock in layers, with spalls behind the arches to give them adequate support and so prevent their failure.

The writer would strongly urge all engineers first to heed the careful words of Mr. Dillman on design; next to examine exhaustively all the materials in the vicinity of any proposed dam that might enter into the structure, eliminating any cheap untried methods of experimenters and scrutinizing carefully vendors of special cheap types; then to select a type of structure that is best suited to the location and service required, regardless of the cost; and then to build so that it will not fail,—not like Lord Macauley's idea of the New Zealand tourist who, to illustrate the instability of things, was to stand on a broken arch of London Bridge and sketch the ruins of St. Paul's (London Bridge and St. Paul's are now being underpinned due to defective foundations in both structures, and the work is being done by British engineers).

J. B. Lippincott,* M. Am. Soc. C. E. (by letter).†—The authors are to be commended for writing so frank a discussion of the failure of the Gem Lake Dam due to the freezing and thawing of the concrete on the outer side of the arches below the water levels in the reservoir. The use of this type of dam in regions subject to severe frost has long been a subject of conjecture among Western engineers. Although a thin shell of concrete may be made water-tight against percolation, it is not possible by any process known to the writer to prevent it from absorbing water when in contact with it. The Gem Lake Dam has shown that this is an unsatisfactory condition, in locations of extreme cold.

The multiple-arch dam because of the economy of its construction has such merit that its future use in many localities is assured. This type of structure, however, has its limitations which enthusiasts, in the past, have sometimes overlooked. Most of the dams of this type have had buttresses of plain concrete without reinforcement.

The writer has had occasion to examine the buttresses of five of these dams and in each has found one or more diagonal cracks in some of the buttresses, starting from the junction with the arch approximately two-thirds down from the top of the dam, and running in a line generally normal to the water face to an intersection of the foundation. These cracks do not extend into the reinforced arch rings. In one dam this type of crack developed prior to the time that the arch rings were built on the buttresses. In another instance the cracks are reported to have appeared after the arch rings were placed, but before the reservoir was filled. They are believed by the writer to be due to shrinkage caused by either the drying of the concrete or by tem-

* Cons. Hydr. Engr., Los Angeles, Calif.
† Received by the Secretary, November 10, 1925.

perature changes, because they open wider in winter and partly close in summer.

It is almost impossible to set up a triangular slab of concrete, say, 100 ft. high, with a base length of 100 ft. firmly anchored into bed-rock without having shrinkage cracks occur. This shrinkage and cracking of flat concrete pavements at intervals of 30 to 50 ft. is common. These cracks do not occur in all the buttresses of the dams examined, but usually in a number of the higher ones. Some cracks of this kind have been noticed, however, in buttresses that are not more than 30 ft. high. In every multiple-arch dam having an age of, say, two years, that has been examined by the writer, such cracks have been found, including one having reinforcement.

It is believed that these cracks do not indicate a failure or any serious weakness of the structure, but they are disturbing and lead to discussion and comment. They distinctly suggest the necessity of thoroughly reinforcing the buttress walls of multiple-arch dams in order to minimize such conditions. This paper does not mention any such occurrence in the Gem Lake Dam, but as attention has been called by the authors to some of the limitations in this type of structure, it is deemed appropriate to call attention also to an improvement of past practice.

OREN REED,* JUN. AM. SOC. C. E. (by letter).†—The writer has followed the fortune of the Gem Lake Dam with a great deal of interest. The Company with which he is associated has plans for two regulating dams on the North Fork of Kings River on the west side of the Sierras, at elevations of 6 550 ft. and 8 170 ft. The failure of the Gem Lake and other dams leaves a doubt in the minds of engineers as to the permanency of such structures when subject to the action of frost and ice. On the other hand, Lake Eleanor Dam,‡ 4 660 ft. above sea-level, of the multiple-arch type, has withstood frost action without damage, and several dams of the Ambursen type have been built in Canada and the northern part of the United States with slight damage from frost.§ The Southern California Edison Company is constructing a multiple-arch dam at Florence Lake on the San Joaquin River, at an elevation of about 7 200 ft.

One wonders whether the primary cause of the failure of Gem Lake Dam was frost action or whether frost action was made possible by a weak porous concrete. It would have been of interest if the authors had stated whether the Lake Agnew Dam had also been damaged by frost, since it is only 550 ft. lower in elevation.

Since information as to the destruction of concrete by acidic and even pure water is very meager, some discussion on that important subject may be of value. While in Europe in 1922-23 the writer was much impressed by the general interest among engineers in this subject. Many tests of concrete, as used in the construction of dams, have been made in Germany, Denmark, and

* Asst. Designing Engr., San Joaquin Light & Power Corporation, Fresno, Calif.
† Received by the Secretary November 16, 1925.
‡ *Engineering News-Record*, Vol. 95, July 20, 1925.
§ *Loc. cit.*, September 3, 1925.

Norway during the last few years.* It was found that some dams had been failures, but as a rule these failures had been of a purely local character and could be laid to poor workmanship and frost, and a few to poor sand, etc. In other cases the weakening had been more general, especially where water had an opportunity to work in porous concrete. There are several factors which were thought to have contributed to these failures.

1.—Sand.—Sand containing dirt, humus or humic acids was found to make poor concrete. Concrete made with humus-holding sand has a comparatively low early strength and must be kept moist while hardening in order to gain its normal strength. These humus materials can be disposed of by heating, and, in part, by washing. The sand used should be clean, sharp, and well-graded. Whenever crushed artificial sand is available or can be produced cheaply enough it should be used.

2.—Cement.—In many cases it was found that the cement was at fault. In almost all concrete construction only pure Portland cement is used. This is composed of about 63% lime (CaO), about 20% silicic acid (SiO_2), about 6% of clay and smaller parts of iron oxide (Fe_2O_3), magnesia (MgO), sulphuric acid anhydrate (SO_3), etc. In hardening after hydration lime is freed, which, together with water, forms lime hydrate ($Ca(OH)_2$); this comprises about 15% by weight of the cement's mass. Lime hydrate is soluble in water and is washed out of the structure by leakage water, and, under the action of the carbonic acid in the air, becomes carbonate of lime. This is the white coat to be found on the structure. By the dissolution of lime hydrate the concrete becomes more porous and the voids are filled with leakage water. When freezing takes place, the sand grains are further loosened. This dissolution of lime takes place most easily when the mixture is lean and when humic acid is present in the water or in the sand. Dissolution of the lime may be prevented by adding to the concrete mixture a material which, reacting with the free lime, forms an unsoluble compound, as a silicate of lime. For this purpose the Germans use trass. This is a pale yellow or gray volcanic rock found in the valley of the Eifel in the Lower Rhine. It is a wholly neutral material, but with lime undergoes a chemical combination, forming an insoluble silicate of lime. Finely ground brick clay has also been used with good results.

A concrete may be made water-tight by using a rich cement mortar, but this is a costly method. The French alumina cement, which is especially rich in clay (Al_2O_3), containing 36%, is very durable against acids.

3.—Water.—Water for concrete must be clean and free from organic matter. From tests it has been proven that a plastic concrete gives better results than a dry mixture. A plastic mixture gives a less porous concrete.

4.—Workmanship.—The fourth factor in concrete placing is workmanship. This must be high grade to insure the proper mixing and placing. Poor workmen can produce poor results with the best materials.

The natural lakes and meadows on the west slope of the Sierra Nevada Mountains are, in effect, marshes. To a great extent they are of glacial origin. In many cases large tracts of meadow or marshy ground are covered by water

* Ing. Bonde, *Teknisk Ukeblad*, No. 14, April 3, 1925, and Ing. Groner, *Teknisk Ukeblad*, No. 10, April 17, 1925.

when a dam is constructed. Many tests, especially in Germany, have been made on these marsh waters.

Most marshes contain more or less sulphur compounds, in the form of gypsum, sulphuric pyrite, or even free sulphuric acid. It is well known that sulphuric acid and sulphate-holding water attack concrete by combining with the lime to form gypsum, which is soluble. Destruction by these sulphuric acid combinations occurs not only to new but to old concrete.

Furthermore, great quantities of carbonic acid are found in marshes, which combines with the lime of concrete to form soluble salts. The free lime is thus seen to be the main point of attack and, to prevent failure of the concrete, the lime must be fixed. Tests on marsh water have given varying results but concrete made with it has become weakened sooner or later in nearly all cases. As a general rule the acids which may be present in a reservoir will be in a very dilute state and will not be effective but, where there are large marshy tracts near the dam, and especially with low-water level in the reservoir, the acids will no doubt have a certain effect on the durability of the concrete.

It is not only the sulphate and acid-holding water which damages concrete. Under certain circumstances, pure water, on account of its high dissolving power, may act equally as effectively as weak acids. The water from the granite regions near the crest of the Sierras will often be nearly pure.

In general, concrete tests are concerned only with strength without regard to density, but in dams density is very important. A dense concrete made of the best materials prevents attack both from weak acids and from pure water. A porous concrete has many points of attack for mechanical and chemical action.

Many multiple-arch and flat-deck dams have been successful in Norway and Sweden. One of the most important of these is Holmevand Dam, A/S Saudefaldene, in South Norway. This is a flat-deck dam of the Ambursen type, and is 740 m. (2 400 ft.) above sea level. Fjergeng Dam, in Central Norway, north of Trondhjem, is also a flat-deck dam. As a protection against frost action, it was provided with a vertical insulating wall between the buttresses on the down-stream side. At the Porjus Dam, in Sweden, just above the Arctic Circle, there are concrete arch spillways 648 ft. in length. The remainder of the dam consists of earth embankments faced with stone. The up-stream side of the crest of the concrete dam is inclined, in order to force the ice to break up and be pushed over the crest. A stone-protected dirt fill, furthermore, has been placed in front of the dam to insure against leakage, as in dams of the Intze type. This spillway structure is a modified Ambursen dam and consists of concrete arches between concrete buttresses. The arches in front, against which the fill rests, have a curved exterior, while those forming the crest of the dam and the down-stream face are covered by a flat concrete deck paved with stone set in the mortar.

When it is guarded against, frost action of itself has no great effect on concrete in dams, but as a secondary factor it can do great damage.

THE IMPROVED VENTURI FLUME

Discussion*

By Messrs. E. S. Lindley and C. E. Carter

E. S. Lindley,† M. Am. Soc. C. E. (by letter).‡—The first mention of this device quoted by the author, and the first in America of which the writer is aware, is the paper by Mr. V. M. Cone, published in 1917.§ In the Panjab, use of the device was initiated by the late Messrs. Stoddard and Harvey of the Irrigation Department some time before 1914.

The object with which the device was primarily developed in the Panjab, was control; that is, to secure a constant relation between discharge and water level above the device, with only a small drop of level through it. Calibration and precise knowledge of the discharge passing were of course also important.

The writer wished to extend the use of the device and use it as a Venturi flume, that is, to measure discharge by means of gauge readings at the throat as well as up stream. In correspondence, Clemens Herschel, Past-President, Am. Soc. C. E., recorded the opinion that this was not really practicable; the writer soon came to agree with him. The attempt to develop the Venturi flume was still proceeding in America. To avoid the confusion that arose over applying that name to the "control section" use of the device, the name, "Standing Wave Flume", was originated instead. It would appear that part of the improvement described by Mr. Parshall is to omit the use of the throat gauge except to show when there is enough drop of water surface to avoid damping the maximum discharge.

It is to be remarked that the principles involved are the same whether the "control section" be produced by constricting the width of the channel, or by raising a hump or broad-crested weir on the bed, or by both in combination.

The workers at this form of device are divided into two schools. One school starts with a design of weir or flume that is convenient to build, and that is an effective control, and then calibrates it experimentally. The other school argues that the well-known broad-crest weir formula generally ascribed to Unwin entails a definite physical condition of flow, that any departure from the rates of discharge given by that formula indicates a state of uncertainty of flow, and that the best course is to determine the limits of form that the work must satisfy in order to discharge according to that formula. The former school produces a new formula (as does this paper) for every new shape tested; the latter claim that each such formula is uncertain of correct application out-

* Discussion of the paper by Ralph L. Parshall, Affiliate, Am. Soc. C. E., continued from December, 1925, *Proceedings*.

† Executive Engr., Indian Public Works Dept., Panjab Irrig. Branch, Care, Messrs. Thos. Cook & Son, London, England.

‡ Received by the Secretary, November 14, 1925.

§ *Proceedings*, Am. Soc. C. E., September, 1925, Papers and Discussions, p. 1340.

side the range of the tests on which it is based, and that they can build a structure of any size which will discharge correctly.

Unfortunately, those who have done the most work on this device in India have not placed their facts and views on record. The writer has made an attempt to fill the gap in the course of a technical paper;* a certain amount of data are also to be found more accessibly in a discussion that he contributed to another paper that bears on the subject.† He does not propose to repeat the whole matter here, but only to outline the main points, leaving those who are sufficiently interested to read the original.

The limits of shape have been ascertained, which a broad-crest weir must satisfy to discharge according to the broad-crest formula. Data are quoted from structures built for discharges up to 8 000 cu. ft. per sec. and up to 6½ ft. depth on the crest. Subject to a few limitations and conditions that will be obvious to practical hydraulicians, and in no way impair the practical utility of the works, they are probably correct within about 1 per cent. Those not built to effect any recovery of head are independent of fluctuations of down-stream levels as long as the drop does not fall short of one-third of the depth on the crest. Those built to work with less head are thus independent as long as the drop does not fall short of one-sixth of the depth on the crest for small structures, while the fraction may be shown to be even smaller for larger works.

A rational weir formula is also given (due to Mr. E. S. Crump of the Panjab Irrigation Department), which indicates the manner in which many weirs and flumes discharge more than the broad-crest formula.

Such weirs and flumes on a small scale, and orifices similarly made to discharge as if they had free falls, are now being largely used in Indian irrigation to eliminate manual control and secure automatically correct distribution of irrigation supplies. This subject is fully dealt with in another technical paper by the writer,‡ which also gives a full bibliography of the subject.

In Mr. Parshall's paper, an accuracy of 5% is claimed for the device described. Greater accuracy is not likely to be achieved as long as angles are allowed in the sides and floor. These relatively loose limits also probably cover a certain amount of dependence on the fluctuation of down-stream levels. In the only observation of which the necessary data are reported, the limit for independence occurred with a 0.19-ft. fall for a 0.86-ft. depth on crest, a fraction of two-ninths, or only little better than the Panjab form without attempting to recover the head. The writer considers the position for taking the throat head unsatisfactory; in fact, the essence of objections to a Venturi flume is that throat gauges are unreliable unless the fall is quite small. The reasons for this are fairly obvious even from theory.

The exponent of W in the formula, given in the paper as 0.026,§ would appear to be a misprint; judged by a few checks of calculations in Table 2,‖

* "Canal Falls and Their Use as Meters," Panjab Eng. Congress, 1925.

† "Measurements of Discharge Over a Rock-Faced Dam", *Minutes of Proceedings*, Inst. C. E., Paper No. 4515, 1924.

‡ "The Application of Modules in Irrigation", Panjab Eng. Congress, 1923.

§ *Proceedings*, Am. Soc. C. E., September, 1925, Papers and Discussions, p. 1342.

‖ *Loc. cit.*, p. 1348.

it would seem that the exponent of H_a in the formula should be nearer 1.6 than 1.52, and that the exponent of W is actually 1.0.

The coefficient, 4.0, in the formula is surprising. Mr. Crump's rational formula, previously alluded to, indicates a maximum possible coefficient of 4.18. In some tests with small weirs intended to give the greatest possible coefficients, the writer was unable to find one greater than 3.9, and that seems to be the largest value given by the late Hamilton Smith, M. Am. Soc. C. E.

A paper of this kind is less convincing than it should be, when no account is given of the means and manner of testing, and only a few of the observed data are included.

C. E. CARTER,* M. AM. SOC. C. E. (by letter).†—A means of measuring accurately the flow of water in irrigation canals and other open channels has long been needed and that fact is well appreciated by those who have ever had anything to do with the delivery of certain definite quantities of water to individual consumers.

Mr. Parshall's paper opens the way for making such measurements of flow within a reasonable degree of accuracy and it is to be hoped that future experiments will extend the scope of those presented to include wider channels, greater depth of flow, and larger rates of discharge.

Believing that the author's formula‡ for discharge,

$$Q = 4\ W\ H_a^{1.522\ W^{0.026}}$$

would be in a more workable form if the function of W could be eliminated from the exponent of H_a, the data presented have been studied with this in view and the formula,

$$Q = 3.90\ W\ H_a^{1.58}$$

has been deduced. The writer believes that this formula gives results that are fully as close to the actual discharges as that of the author for the sizes of flumes and quantities of discharge shown.

Using this formula, Table 2§ has been recomputed as shown in Table 3. Only ten items show deviations greater than those by the author's formula, of which only one varies more than 5% from the actual quantity discharged. The remaining thirty-eight items show deviations less than these by the author's formula.

A comparison of the percentages of deviation show (Table 4) that 75% of the computed quantities, using the writer's formula, are within 2% of the observed quantities and that 96% vary less than 5% from the observed quantities.

Although the 1-ft., 2-ft., and 3-ft. flumes show discharges under heads as low as $H_a = 0.20$ ft., the 4-ft., 6-ft., and 8-ft. flumes do not show discharges for heads lower than $H_a = 1.0$ ft. In order to compare the two formulas for heads lower than 1.0 ft. controlling computations have been made, giving the results shown in Table 5.

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† Received by the Secretary, November 16, 1925.

‡ *Proceedings*, Am. Soc. C. E., September, 1925, Papers and Discussions, p. 1342.

§ *Loc. cit.*, p. 1348.

TABLE 3.—COMPARISON OF CALCULATED FREE FLOW WITH OBSERVED DISCHARGE FOR SUBMERGENCY FROM 69 TO 79 PER CENT.

(Based on formula, $Q = 3.90\ W\ H_a^{1.58}$.)

Test No.	H_a. in feet.	H_b. in feet.	Percentage of submergence.	Observed discharge, in cubic feet per second.	Calculated discharge, Carter formula.	Difference, in cubic feet per second.	Percentage of deviation, by Carter formula.	Percentage of deviation by Parshall formula.
				1-FOOT FLUME.				
6707	0.200	0.141	70.5	0.31	0.31	0.00	0.00	12.9
6703	0.398	6.276	69.3	0.91	0.91	0.00	0.0	7.7
6667	0.600	0.424	70.7	1.74	1.74	0.00	0.0	5.7
6699	0.599	0.448	74.8	1.71	1.72	0.01	0.6	7.6
6695	0.801	0.568	70.9	2.72	2.74	0.02	0.8	4.8
6694	0.802	0.605	75.4	2.69	2.75	0.06	2.2	6.3
6710	1.002	0.708	70.7	3.86	3.91	0.05	1.3	3.9
6689	1.001	0.759	75.8	3.78	3.91	0.13	3.4	6.1
6686	1.202	0.857	71.3	5.10	5.22	0.12	2.4	3.7
6675	1.603	1.200	74.9	7.73	8.22	0.49	6.3	6.1
				2-FOOT FLUME.				
6645	0.201	0.140	69.7	0.62	0.62	0.00	0.0	6.5
6646	0.201	0.154	76.6	0.61	0.62	0.01	1.6	8.2
6637	0.402	0.281	69.9	1.87	1.85	−0.02	−1.1	4.3
6638	0.402	0.298	74.1	1.82	1.85	0.03	1.6	7.1
6631	0.598	0.421	70.4	3.49	3.46	−0.03	−0.9	3.2
6632	0.602	0.456	75.7	3.38	3.50	0.12	3.6	7.7
6606	1.002	0.707	70.6	7.89	7.82	−0.07	−0.9	1.7
6607	1.000	0.768	76.8	7.70	7.80	0.10	1.3	3.9
6612	1.602	1.127	70.3	16.51	16.42	−0.09	−0.5	0.6
6613	1.593	1.217	76.4	15.89	16.28	0.39	2.5	3.7
				3-FOOT FLUME.				
6579	0.199	0.140	70.4	0.92	0.93	0.01	1.1	4.3
6575	0.298	0.221	74.2	1.71	1.73	0.02	1.2	5.8
6528	0.399	0.298	74.7	2.75	2.74	−0.01	−0.4	3.6
6520	0.503	0.379	75.4	3.93	3.94	0.01	0.3	4.1
6534	0.595	0.446	75.0	5.13	5.15	0.02	0.4	3.7
6556	0.600	0.466	77.7	5.18	5.22	0.09	1.7	5.1
6514	0.699	0.499	71.4	6.74	6.64	−0.10	−1.5	1.6
6553	0.900	0.647	71.9	10.09	9.91	−0.18	−1.7	0.8
6543	0.996	0.710	71.3	11.84	11.68	−0.11	−0.9	0.8
6561	0.999	0.769	77.0	11.92	11.68	−0.24	−1.2	0.5
6563	1.000	0.754	75.4	11.81	11.70	−0.11	−0.9	1.6
6544	1.004	0.799	79.6	11.79	11.77	−0.02	−0.2	2.5
6572	1.200	0.836	69.7	15.99	15.60	−0.39	−2.4	−0.2
6411	2.039	1.509	74.0	35.23	36.06	0.83	2.4	3.9
6408	2.358	1.697	72.0	45.66	45.38	−0.28	−0.6	0.7
				4-FOOT FLUME.				
6399	1.014	0.791	78.0	15.09	15.95	0.86	5.7	8.4
6395	1.339	0.977	73.0	24.55	24.74	0.19	0.8	3.3
6373	1.702	1.293	76.0	35.44	35.14	0.70	2.0	4.5
6382	2.008	1.475	73.5	46.31	46.93	0.62	1.3	3.8
				6-FOOT FLUME.				
6359	0.900	0.678	75.3	20.11	19.81	−0.30	−1.5	0.9
6353	1.143	0.803	70.3	30.04	28.90	−1.14	−3.8	−1.1
6354	1.171	0.905	77.3	29.94	30.03	0.09	0.3	3.1
6347	1.396	1.065	76.3	40.20	39.64	+0.58	+1.4	1.6
6339	1.562	1.120	71.7	49.51	47.34	+2.17	−4.4	−1.3
6340	1.617	1.273	78.7	50.07	50.00	−0.07	−0.1	3.2

TABLE 3.—(*Continued.*)

Test No.	H_a, in feet.	H_b, in feet.	Percentage of submergence.	Observed discharge, in cubic feet per second.	Calculated discharge, Carter formula.	Difference, in cubic feet per second.	Percentage of deviation, by Carter formula.	Percentage of deviation, by Parshall formula.
				8-Foot Flume.				
6313	1.064	0.774	72.7	35.08	34.44	—0.64	—1.8	0.8
6304	1.246	0.871	69.9	45.12	44.15	—0.97	—2.1	1.0
6305	1.270	0.937	73.8	45.23	45.41	0.18	0.4	3.8

TABLE 4.—Comparison of Deviations from Observed Discharges.

Percentage of deviation.	Percentage of Deviation.	
	Carter formula.	Parshall formula.
1 or less	42	19
2 " "	75	31
3 " "	88	33
4 " "	94	60
5 " "	96	71

TABLE 5.—Comparison of Formulas for Low Heads on the 4-Foot, 6-Foot, and 8-Foot Flumes.

Width of flume, in feet. (1)	H_a, in feet. (2)	Computed Discharge, in Cubic Feet per Second.			Percentage of variation, $\frac{\text{Column (5)}}{\text{Column (3)}} 100$.
		Parshall formula. (3)	Carter formula. (4)	Difference. (5)	(6)
4	0.20	1.27	1.24	—0.03	—2.5
	0.60	7.15	6.96	—0.19	—2.6
	1.00	16.00	15.60	—0.40	—2.5
6	0.20	1.84	1.86	0.02	1.1
	0.60	10.63	10.44	—0.17	—1.6
	1.00	24.00	23.40	—0.60	—2.5
8	0.20	2.41	2.48	—0.07	2.9
	0.60	14.09	13.92	—0.17	—1.2
	1.00	32.00	31.20	—0.80	—2.5

A study based on the complete record of experiments for all the flumes might show that a slight change in the constant or exponent could be used to advantage, but from the foregoing, the writer would expect that the formula,

$$Q = 3.90\ W\ H_a^{1.58}$$

would give results well within 5% of actual discharges for all sizes of flumes and heads within the scope of the experiments.

SIDE CHANNEL SPILLWAYS: HYDRAULIC THEORY, ECONOMIC FACTORS, AND EXPERIMENTAL DETERMINATION OF LOSSES

Discussion*

By Messrs. I. Gutmann and A. J. Wiley

I. Gutmann,† Assoc. M. Am. Soc. C. E. (by letter).‡—The following is a partial list of dams with spillways of the type discussed in Mr. Hinds' paper:

	Date of construction.		Date of construction.
Solingen, Germany	1902	Ashokan, New York City Water Supply	1912
Queille, France	1904	Mauer, Germany	1912
New Croton, New York City Water Supply	1906	Oakley, Idaho	1913
Wachusett, Boston, Mass., Water Supply	1906	Moehne, Germany	1913
Rochebut, France	1909	Klingenberg, Bohemia	1914
Cold Springs, Oregon	1910	Cedar Lake, Seattle, Wash., Water Supply	1915
Conconully, Washington	1910	Morena, San Diego, Calif., Water Supply	1915
Croton Falls, New York City Water Supply	1910	Arrowrock, Idaho	1915
Sweetwater, San Diego, Calif., Water Supply	1911		

This list shows that the side channel type of spillway is extensively used in the United States as well as in Europe. In fact, it is usually the most appealing solution in cases where the dam site is in a canyon and a properly located saddle in the rim of the reservoir is not available.

Mr. Hinds' paper presents, as far as the writer knows, the first systematic analysis of this subject in American technical literature. The paper is confined principally to the analysis of one characteristic of flow in side channel spillways, namely, that the total discharge at any section is a linear function of the distance, x, from the head of the channel or, in mathematical terms, $Q = bx$.

It may be stated that the type of discharge expressed by this equation is not characteristic of side channel spillways only. In general, it applies also to the discharge of a drain passing through uniformly porous soil, an infiltration gallery through uniform gravel, etc., and by changing its sign to read, $Q = -bx$, it will give the equation of flow in a channel subject to uniform seepage losses.

* Discussion of the paper by Julian Hinds, M. Am. Soc. C. E., continued from December, 1925, *Proceedings*.

† With T. H. Wiggin, Cons. Engr., New York, N. Y.

‡ Received by the Secretary, November 9, 1925.

Some of the specific and very perturbing problems in the design of side channel spillways are not discussed in Mr. Hinds' paper. The writer has in mind particularly the phenomena caused by the change in direction of flow of the over-falling nappe from normal to axial with reference to the weir. It is hoped that the observations and experiments described by Mr. Hinds will be extended to include studies of the transverse surface curve in the spillway channel, the extent of the effective area of the spillway channel, hydraulic jump, and other phenomena arising from the change in direction of flow.

The writer wishes to call attention to an oversight in the preamble to Equation (17),* $\frac{dA}{dd}$ is equal T only in the case of rectangular channels and, therefore, Equations (17) and (18)* will hold only for channels of that cross-section.

A. J. WILEY,† M. AM. SOC. C. E. (by letter).‡—Mr. Hinds has given a complete and clear exposition of the hydraulics of the side channel spillways that have been used in a number of important dams.

Often the spillway is the most difficult part of a dam to design and one of the most expensive to construct. The side channel spillway, as Mr. Hinds correctly states, is best suited for moderate discharges, but it has been used for flood discharges of 75 000 sec-ft. Unless the protected channel is extended to deliver the water back into the river, the concentration of such a large volume in a comparatively small channel, with the resulting high velocities, may cause extensive erosion the deposits from which disturb the river flow below the dam. The channels of the Arrowrock and the Don Pedro Spillways both end a few hundred feet below the dam in a vertical drop over the unprotected canyon slopes from which the water finds its own way back into the river.

The formation at Arrowrock at the end of the channel is a rather soft granite. The erosion was quite rapid at first and the resulting deposits caused a local rise in the river bed. It soon created a vertical fall at the end of the lined channel from which the discharge flows in an eroded channel which joins the river several hundred feet below the toe of the dam. The conditions are stabilized for ordinary flood conditions, and future erosions except for extreme flood discharges will be negligible.

At the Don Pedro Dam, where the drop from the reservoir surface to the river below the dam is about 275 ft., the rock is a very hard diorite with many surface seams. At first there was quite extensive erosion of the large blocks of seamy surface rock with a discharge of 15 000 sec-ft., the deposit of which in the river channel caused a rise of about 20 ft. in the back-water at the dam. In this case, as in the Arrowrock Dam, the conditions soon became stabilized and further erosion will be negligible for ordinary flood discharges.

* *Proceedings*, Am. Soc. C. E., September, 1925, Papers and Discussions, p. 1360.

† Cons. Hydr. Engr., Boise, Idaho.

‡ Received by the Secretary, November 28, 1925.

The spillway of a storage dam discharges only a small part of the river flow, the major part being drawn off through the regulated discharge of the irrigation or power outlets; but it is necessary to make spillway provision for the maximum unregulated flood discharges.

The chief objection to the channel type of spillway is the difficulty of protecting the channel lining against upward pressure. Usually the channel is cut out of the solid rock, beginning, perhaps, 500 ft. above the dam and is about 20 ft. wide and 20 ft. deep. It gradually increases in dimensions until, when it passes the end of the dam, it is, in the case of the Don Pedro Dam, 70 ft. wide and 50 ft. deep. Concrete lining is necessary to avoid erosion at the high velocities obtaining, but the cost of the thickness required to resist the full upward pressure from the reservoir is prohibitive and dependence must be placed on under-drainage to carry off the water that finds its way through the seams in the rock, the quantity of which is uncertain. In the Don Pedro Spillway channel, a part of the floor was raised by the excess of the under-pressure over the relief provided. The concrete lining was replaced and about 450 bbl. of cement were used in pressure grouting the rock floor of the channel although it was from 20 to 50 ft. below the surface of the rock excavation.

In many cases the safest and cheapest spillway is directly over the top of the dam. This form is embodied in the design of the Melones constant radius arch dam, 200 ft. high above the river level, and now under construction. In many storage dams, an important source of revenue is found in the development of water power from the flow as regulated for irrigation. As this is "seasonal" power, it must be cheaply developed in view of its low market value. The lowest development cost usually results from building the power plant directly at the toe of the dam, as at the Don Pedro Dam. At the Exchequer Dam, now nearing completion, where the river is raised 325 ft. above its old level, the power plant is built in the river channel at the down-stream toe. A modification of the side channel spillway is made by converting each end of the regular gravity type dam into an overflow section. These two overflow sections take the place of the single long lip, or weir, above the dam which feeds the ordinary type of spillway channel. The discharge from each of the overflow sections of the dam is carried in a concrete-lined channel, symmetrically located on either side of the canyon, leaving the toe of the dam at the end of the overflow section and discharging into the river down stream from the power house.

Instead of determining the hydraulics of the spillway channel by the method outlined by Mr. Hinds, the writer has been accustomed to use a much less direct method; instead of adopting the most economical shape, he has used a broader and shallower section in which the velocities are relatively low and the danger of uplift under the floor is reduced.

Starting with a considerable submergence of the spillway lip at the upper end, assumed velocities and bottom widths are adopted, increasing by regular increments along the spillway section of the channel. Using assumed side slopes the depth at any point is calculated for the quantity of water at the point with the assumed velocity. The fall in water surface necessary to give

the assumed velocity is found by considering that the velocity at the first of any two successive points is reduced at the second point to the same proportion of that at the first point as the discharge at the first point is to that at the second point. The head required to increase the velocity thus reduced up to the required velocity at the second point plus the sum of the velocity heads and friction heads of the first point plus the friction head between the two points then gives the total fall in water surface up to the second of the two points. To provide for the increase in volume by entrained air and for the losses caused by shock and eddies, an arbitrary addition of 25% is made to the required flood discharge.

PERMISSIBLE CANAL VELOCITIES

Discussion*

By Messrs. R. A. Hill, Jerome H. Fertig, E. S. Lindley, and Andrew Weiss

R. A. Hill,† Assoc. M. Am. Soc. C. E. (by letter).‡—The very careful distinction which the authors make between canals carrying colloidal matter and those carrying only clear water or materials not colloidal in nature is one of especial value and in itself warrants all the efforts spent in gathering and interpreting the data.

Consideration of the effect of infiltration of colloidal matter into the sides and bottoms of canals has been disregarded too many times in the development of irrigation projects in the Southwest. Equally unfortunate has been the experience of those familiar only with canals carrying colloidal material when they are called on to design irrigation works for other conditions. As the writer's experience has been very largely with irrigation canals served from silty rivers, he will limit his discussion to this particular phase of canal velocities.

While the conditions found by Mr. R. G. Kennedy in Northern India probably are considerably different from those in America, it would seem that in dismissing his conclusions as fallacious, the authors have overlooked the fact that any relation found reasonably applicable there should likewise, when properly modified, be reasonably applicable to canals in those parts of the United States that have climatic conditions generally similar to Northern India; furthermore, that the so-called critical velocity is really not a single velocity but rather a range of velocities between which neither silting nor scouring will occur.

Where the material being transported by the water in canals is not colloidal in nature the range of velocity between which silting will cease and scouring begin will become smaller as the particles comprising the bed of the channel approximate the size of the particles carried in suspension. In practically all cases, however, the bed of the channel is composed either of colloidal material or of particles materially larger than those carried in suspension. Consequently, it should be obvious that a velocity which will not cause silting need not cause scour or erosion.

While the relation proposed by Mr. Kennedy, that the velocity should be proportional to the 0.64 power of the depth, would be applicable only in rare

* Discussion of the paper by Samuel Fortier and Fred C. Scobey, Members, Am. Soc. C. E., continued from December, 1925, *Proceedings*.

† Associate, Quinton, Code & Hill, Los Angeles, Calif.

‡ Received by the Secretary, October 22, 1925.

instances, nevertheless there must exist in any channel a relation between the depth and the permissible velocity. It is generally recognized on irrigation projects where silty water must be handled that a deep channel is far more apt to silt than a channel having an equal mean velocity but a less depth. This recognition on one project took the form of an empirical rule that the depth should be one-thirteenth of the bottom width. Analysis of this "rule-of-thumb" will show that when the depth is increased the hydraulic characteristics will be such, under ordinary circumstances, that the velocity will also increase. On other projects, attempts have been made to follow Kennedy's rule or some modification thereof.

If, for the sake of illustration, it is assumed that water is diverted from a silt-carrying river into two large canals similar in capacity and of equal area, one of which is, however, twice the depth of the other, it will be seen that consideration must be given to this factor of depth. At the head of each channel the same quantity of silt is present at all times. The sides of most artificial channels carrying silty water are rather steep, consequently the bottom width of these canals must be approximately in the ratio of 1 to 2; furthermore, the bottom width will be a large percentage of the wetted perimeter.

In order to prevent this silt from falling by the force of gravity to the bed of either channel, eddies must be created to raise the silt toward the surface. About the same number of eddies of equal intensity would be required to keep the silt in the deeper channel from settling to the bottom as would be required in the broader and shallower channel. As the deeper channel is only one-half the width of the shallower channel, there would have to be about twice as many eddies per unit of wetted area, or the eddies would have to be of double intensity in order to maintain the silt in suspension. These eddies can only be created by the energy of the water in contact with the bed of the channel, consequently, in order to develop twice the number of eddies or eddies of double intensity over an equal area, it must be necessary to increase the velocity in the channel if the depth of the channel is increased.

Although the most efficient hydraulic section is that in which the bottom width is roughly equal to the depth, no river naturally forms such a channel, the tendency being quite opposite in that the bottom width is usually from 10 to even 100 times the depth. Rivers carrying sediment and flowing between banks of unstable material naturally adapt themselves to a cross-section in which only local silting or scouring will occur so long as the velocity is fairly constant. On such streams bottom scour will commence as soon as the river rises and will continue until a depth has been reached which is sufficient to maintain equilibrium between the tendency to deposit suspended matter and the tendency to erode material from the bed of the channel. As the river recedes the channel gradually refills until equilibrium is again reached at some low stage.

Such fluctuations in the depth of the Colorado River are a most fortunate illustration of the effect of depth on the silt-carrying capacity of a stream. On Fig. 1 is shown the actual relations between depth and mean velocity, and depth and percentage of silt in the Colorado River at Yuma, Ariz. It will be

noted that as the velocity increases rapidly the percentage of silt held in suspension also increases rapidly, but when the velocity becomes approximately constant, the percentage of silt steadily decreases as the depth increases. When the mean depth is about 10 ft. the silt content is normally about 1.1%, but when the depth is 30 ft. the silt content is only about 0.4%, although the mean velocity is practically constant throughout this range in depth.

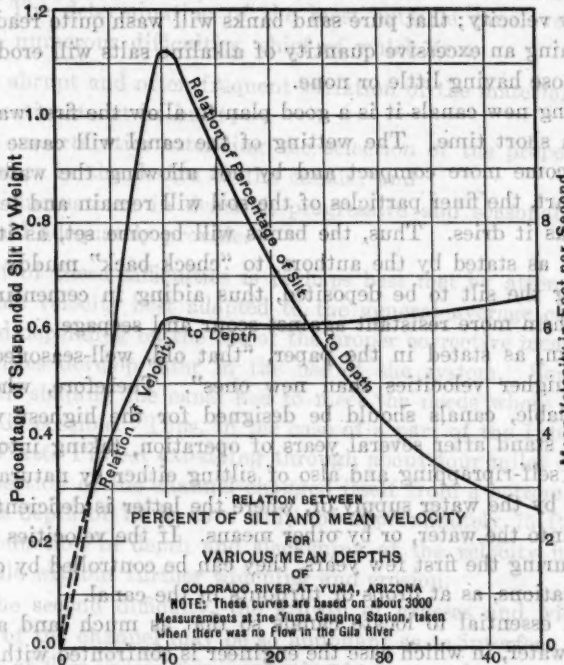

FIG. 1.

Conversely, if silting of artificial channels is to be avoided, the percentage of suspended matter carried by the water must be held constant, hence the mean velocity must be in accord with some increasing function of the depth of the channel. Since bottom scour cannot take place except at velocities higher than non-silting velocities, consideration must be given to the depth at which the canal will normally be operated, otherwise the maximum velocity ordinarily considered safe against scouring might be insufficient to prevent silting of the deep canals required to carry very large quantities. As pointed out by the authors, bank scour is of little consequence due to the berm which naturally forms.

It is hoped that the authors, through the Society's Special Committee on Irrigation Hydraulics, will continue their studies to include the effect of depth on silting of irrigation channels.

JEROME H. FERTIG,* M. AM. SOC. C. E. (by letter).†—The authors are to be commended on the able manner in which they have presented the subject and their conclusions.

It appears that the table of permissible velocities submitted by the writer‡ is somewhat conservative. This is due, possibly, to the fact that his experience has been largely in the construction of irrigation works and in the operation of such works only during the first year or two. His observation has been that new canals and laterals in earth will sustain without scour only a comparatively low velocity; that pure sand banks will wash quite readily; and that shales containing an excessive quantity of alkaline salts will erode much more easily than those having little or none.

In seasoning new canals it is a good plan to allow the first water turned in to run only a short time. The wetting of the canal will cause the banks to settle and become more compact and by not allowing the water to run too long at the start, the finer particles of the soil will remain and tend to cement the material as it dries. Thus, the banks will become set, as it were. It is also desirable, as stated by the authors, to "check back" muddy water in the canal allowing the silt to be deposited, thus aiding in cementing the banks and making them more resistant against scour and seepage.

It is known, as stated in the paper, "that old, well-seasoned canals will stand much higher velocities than new ones". Therefore, where sufficient grade is available, canals should be designed for the highest velocities the materials will stand after several years of operation, taking into account the probability of self-riprapping and also of silting either by natural silt carried into the canal by the water supply or, where the latter is deficient, by sluicing fine material into the water, or by other means. If the velocities are too high, particularly during the first few years, they can be controlled by checks placed at suitable locations, as at drops or turnouts in the canal.

It is often essential to locate canals so that as much land as possible is supplied with water, in which case the engineer is confronted with the question of the minimum allowable velocity rather than the maximum. This, of course, presumes a thorough study of costs of larger canals, and the position and elevation of the intake, the latter often being rather definitely fixed because of controlling topography.

The writer feels that the authors have handled the subject ably and desires to express his accord with their views.

E. S. LINDLEY,§ M. AM. SOC. C. E. (by letter).‖—The authors quote¶ the Kennedy formula for critical velocities, but what is said gives the impression that the original paper has not been fully understood; and it shows an inclination to generalize even outside the field that Kennedy only glanced at tentatively.

* Los Angeles, Calif.
† Received by the Secretary, October 27, 1925.
‡ *Proceedings*, Am. Soc. C. E., September, 1925, Papers and Discussions, p. 1404.
§ Executive Engr., Indian Public Works Dept., Panjab Irrig. Branch, London, England.
‖ Received by the Secretary, November 14, 1925.
¶ *Proceedings*, Am. Soc. C. E., September, 1925, Papers and Discussions, p. 1398.

Twenty years elapsed before any concrete attempt was made to advance from Kennedy's pioneer work. In a few Panjab technical papers* various aspects of the problem have been attacked, and matters pertinent to the present discussion are given. In the course of these papers and the ensuing discussions, reference is made to most of the work known in the East.

ANDREW WEISS,† M. AM. SOC. C. E. (by letter).‡—The excellent report on the subject of permissible canal velocities prompts the following supplementary comment. The determination of the most suitable canal velocities is associated with numerous difficulties, chief of which are:

1.—The abrupt and often frequent variation of the material through which the canal section must be built;

2.—The uncertainties attending the selection of the proper coefficients or friction factors in the design of the canals; and

3.—The fluctuations in demand, progressive and seasonal, for which the system should adequately provide.

The first of these difficulties is perhaps best met by attempting to obtain some suitable velocity best adapted to the general average conditions of the locality, and depending on the use of the proper corrective measures as unforeseen difficulties develop later in the use of the system. Sometimes Nature will assist in shaping the canal bed to meet the needs where such an adjustment is made possible. Thus, in the case of a part of the Interstate Canal of the North Platte Project extending through about four miles of shifting dune sands, the section of the channel changed itself from a bottom width of 22 ft. and a water depth of 10 ft. to a width of approximately 75 ft., with a corresponding reduction in depth and slackening of the velocity until the section became stable without further widening and erosion.

As to the second difficulty, the growth of grasses and willows along the water edge of the channel may be so abundant as to interfere materially with the velocity of flow unless this vegetation is frequently removed with a corresponding increase in the cost of maintenance; on the other hand, the velocity may be greater than expected by reason of a friction factor less than that assumed in the design of the canal. Measurements of velocity on the Interstate Canal have indicated a value of n in Kutter's formula generally less than 0.020 and, in some instances, as low as 0.017, whereas in the design of the canal the coefficient 0.025 was used. The result of this was that erosion started immediately when the canal was put into operation, at times with serious consequences. In addition, its discharge was only about two-thirds its rated capacity. The immediate effect of the excess velocity was the development of sand-boils in the canal bed, on the outside of curves, which were followed by the rapid undermining and destruction of the adjacent canal bank. This difficulty is more serious on the larger canals, because of the relatively greater

* "The Design of Head Regulators," and two other papers, Panjab Eng. Congress, 1916; "Régime Channels," Panjab Eng. Congress, 1919; and "Economic Sections of Kennedy Channels", *Panjab Irrigation Review*, 1925.

† Asst. Director of Farm Economics, U. S. Bureau of Reclamation, Denver, Colo.

‡ Received by the Secretary, November 28, 1925.

rapidity of the development of these boils and the disproportionately large damages which may result from a breach of the canal.

This difficulty was successfully overcome by the use of check structures in the canal bed, which served as a means to control velocities and proved almost indispensable in regulating the diversions from the canal.

The third difficulty, namely, the fluctuation in the demand for which the system may be constructed, is due to seasonal causes as well as to the conditions which may prevail during the period of development of any system. If such development proceeds slowly on account of agricultural depression or other causes, the system may be only partly used for several years. There are also seasonal variations in the use of any canal system which are especially notable in the Northern latitudes, and which bring about a sharp rise in the demand on the system during June, or more often in July, according to local weather conditions and the variety of crops predominating.

As a result of the observations made on the North Platte Project the following conclusions have been reached:

(*a*) With the fine silt coating resulting from the use of the muddy waters from the direct flow of the North Platte the value of n in Kutter's formula should not exceed 0.020, except on the smaller canals or distributaries on which the growth of long grasses and willows along the water edge may obstruct the velocity of the water to a considerable extent, thereby necessitating more frequent cleanings and consequent larger expense for the proper maintenance of the system. Similar and more serious obstructions may result from the growth of aquatic plants in clear water as noted in the paper. For these reasons the use of the larger friction factor, 0.025 (Kutter's formula), is favored in the design of these smaller canals.

(*b*) Owing to the frequent changes of the structure of the soil through which these canals had to be built, it was best to attempt to obtain some suitable average friction coefficient and resultant velocity. In the case of the larger canals velocities ranging from 2.50 to 2.75 ft. per sec. were sought and on the distributaries, velocities of about 0.5 ft. per sec. greater.

(*c*) In all cases the best ultimate results were obtained by the use of check structures in the canals and distributaries, these serving the dual purpose of controlling velocities and facilitating the regulation of diversions. It was found to be practically impossible to perform these functions without check structures. By this means provision was made for these uncertain factors and conditions which could not be foreseen in advance of the construction and operation of these canals.

(*d*) In the design of the check structures it is desirable to use a simple method of closure, preferably check planks, all or any part of which may be removed to correspond to the fluctuations in use and the demands incident thereto. By this means the removal of the planks will restore the original canal velocities if such is found desirable during certain periods, as when it is necessary to sluice accumulated material, or after the canal has become seasoned, or when for other reasons it is not desired to maintain these structures in active use.

(*e*) It is usually desirable to adopt a velocity somewhat in excess of the ultimate rate required because after some period of use the growth of obstructing grasses and willows may cause a considerable retardation of the flow, embarrassing shortages in capacity and additional cost of maintenance of the system. This suggests that a reasonable surplus capacity in the design of such channels may be desirable and economical if the cost of operation and maintenance is given due consideration.

The range of velocities indicated in Table 6* appears to be well applicable in all cases with which the writer has come in contact. There are occasional instances where it may be desirable to avoid the higher velocities in order to induce the deposition of silt for the purpose of reducing percolation losses.

* *Proceedings*, Am. Soc. C. E., September, 1925, Papers and Discussions, p. 1411.

THE ORADELL DAM OF THE HACKENSACK WATER COMPANY

Discussion*

By Messrs. Stuart K. Knox, Howard T. Critchlow, and A. T. Ricketts

Stuart K. Knox,† M. Am. Soc. C. E.—It would be difficult to add anything of importance to the clear description of the design given by Mr. Hill.

The most interesting part of the Oradell Dam is the spillway, which extends throughout the greater part of its length. This is an earthen section protected on its down-stream side by a concrete envelope, so that it may be overtopped without injury. It is this concrete envelope with its support of round piles and its steel sheet-pile cut-off, which constitutes the novel feature of the design.

When the plans for the dam were prepared, about fifteen years ago, the ideas were supplied by Mr. Hill. The speaker's function was merely that of developing the author's notions and in working out details. He recalls that when this particular idea was suggested, it did not, at first, appeal to him. His first impression of the proposed construction was that of a reinforced concrete dam perched on stilts, in a somewhat precarious position; of a buttressed dam turned hindside before. He feared the difficulty of insuring the structure against undermining, and that its purpose would thereby be defeated.

However, these unfavorable first impressions were quickly removed after a detailed analysis had demonstrated the practicability of the construction proposed. After careful examination of the stresses and of the resultant forces to which the dam would be subjected under the most extreme conditions of loading that could be imagined, he became convinced that the design represented an intelligent and effective solution of a really difficult problem, and that is his opinion at present.

The actual conditions of loading to which the concrete envelope would be subjected were unusually difficult to predetermine. Three or four conditions were assumed, therefore, which it was thought represented the greatest possible extremes. The stresses and resultant forces were computed on the assumption that the full water pressure had been applied to the concrete envelope itself, without any earth fill, a condition which might conceivably have arisen after the concrete envelope had been completed and before the back-fill had been placed. They were also computed on the basis of the back-fill having been placed before the water pressure was applied. Sets of calculations were made

* This discussion (of the paper by Nicholas S. Hill, Jr., M. Am. Soc. C. E., published in October, 1925, *Proceedings* and presented at the meeting of November 4, 1925), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† (Nicholas S. Hill, Jr.), New York, N. Y.

on the assumptions that there was, and that there was not, upward water pressure on the bottom of the concrete envelope.

The stresses were first computed graphically and finally were carefully checked analytically. The resultant of the forces under each of the various assumed conditions of loading cut the base of the envelope close to its center.

HOWARD T. CRITCHLOW,* ASSOC. M. AM. SOC. C. E. (by letter).†—In 1912 the State of New Jersey passed a law (Chapter 243, P. L. 1912) authorizing the State Water Supply Commission to have supervision over the erection and maintenance of dams on certain rivers and streams or reservoirs within the State or between it and any other State, in order to protect life and property which might be endangered by the improper construction of any dam and reservoir. In 1916, the Board of Conservation and Development became the successor of the Commission. As Hydraulic Engineer of the Department the writer was called on to examine the plans for the Oradell Dam on the Hackensack River.

On account of the unique design and the importance of the structure, the Board appointed a committee of experts, consisting of William H. Burr, M. Am. Soc. C. E., Arthur H. Pratt, M. Am. Soc. C. E., and the writer, to examine the plans and specifications and determine whether they were in accord with good engineering practice. It is to the credit of the author and his associates that the Committee of Experts found so little to criticize in the plans for the structure. After careful consideration of the foundation materials immediately beneath the concrete envelope, the Committee recommended that protecting paving below the spillway and sluice-gate sections be extended farther down stream in order to protect the structure against scouring of the unconsolidated materials on which it rested. It was also pointed out that, inasmuch as the sluice-gate section was designed to handle a large part of the flood waters, the gates must have adequate attendance to insure that they may be opened promptly and be kept clear of ice and floating débris which might cause plugging of the openings or fouling of the gate mechanism.

In checking the capacity of the sluice-gates and spillway to pass extreme floods, a rate of 100 sec-ft. per sq. mile was considered. With this assumption the head on the spillway with all sluice-gates open would be about 2.2 ft. Although this head is 1.2 ft. higher than that assumed by the author the dam can safely sustain such a flow with all the gates open. The author bases his assumption of maximum flood flow of 71 sec-ft. per sq. mile largely on data in Table 1‡ which is given in millions of gallons per day instead of the usual second-feet. Some of these records do not represent the maximum floods. The discharge of the Pequannock River at Macopin Dam (drainage area, 62.5 sq. miles) is recorded‡ as 201 350 000 cu. ft. in 10 hours from 10:00 P. M., October 9, to 8:00 A. M., October 10, 1903, which represents an average discharge of 88 sec-ft. per sq. mile for 10 hours. What may have been the maximum rate is not recorded, but it must have been considerably more than 100 sec-ft. per sq. mile.

* Chf., Div. of Waters, Dept. of Conservation and Development, Trenton, N. J.

† Received by the Secretary, November 20, 1925.

‡ *Water Supply Paper No. 92*, U. S. Geological Survey, p. 16.

In examining the stability and safety of the Pompton Lakes Dam on Ramapo River, New Jersey, which was built in 1908 prior to State supervision, the writer had occasion to study the flood records on this stream. The following is quoted from his official report:

"The drainage area of the Ramapo at the confluence with the Pompton River is 160.7 sq. miles, while at the Pompton Lakes Dam 1.5 miles above the mouth, the drainage area is 159.5 sq. miles. In Volume III of the New Jersey Geological Survey (1894), page 155, it is stated that during the flood of September, 1882, the Ramapo had a maximum discharge of 10 540 cu. ft. per sec., which is equivalent to a water level in the lake of about 4.5 ft. above the elevation of the crest of the dam. In the U. S. Geological Survey *Water Supply Paper No. 88*, page 35, it is stated that during the flood of February-March, 1902, 'on the Ramapo River at Pompton Furnace the highest water was observed at 6:00 P. M. on March 1, when it reached 4.1 ft. above the dam crest'. In the Annual Report of the New Jersey Geological Survey for 1903, pages 27 and 28, it is stated that during the flood of October, 1903, 'the Ramapo at about 5:40 A. M. of October 9, when the Pompton Steel Works dam broke, had reached a discharge of 7 123 cu. ft. per sec. * * * Undoubtedly the maximum discharge was much in excess of this.' This same report in Table V, on page 27, states that the maximum discharge of the Ramapo during the flood of 1896 was 8.731 cu. ft. per sec. Thus it is seen that during a period of less than 25 years there were 4 floods that caused a lake level of over 4 ft. above the crest of the dam and a possibility that during the flood of 1903 the level would have been 5 ft. above the crest if the dam had not broken. [The crest of the old dam was of the same elevation as the existing structure.] Therefore, in analyzing the strength of the dam and wing-wall a flood level 5 ft. above the crest of the dam has been considered."

A 5-ft. head on the Pompton Lakes Dam (300 ft. long) is equivalent to a discharge of 85 sec-ft. per sq. mile, as compared with the record of 66 sec-ft. per sq. mile given in Table 1 for the flood of September, 1882. The flood of October, 1903, appears to have been a more severe flood than that of September, 1882, on North Jersey streams. Unfortunately, few permanently established stream-gauging stations were being operated during that flood. There are now being maintained in New Jersey thirty-six permanent gauging stations, twenty-six of which are equipped with continuous water-stage recorders. No extreme flood heights have yet been recorded. The heights given in Table 2 are, however, of record.

TABLE 2.—RECORDED RUN-OFFS.

Name of stream and location.	Drainage area, in square miles.	Maximum flood flow, in second-feet per square mile.	Date.
Swimming River, near Red Bank, N. J....	47.7	80	July 23, 1919
Lawrence Brook, at Weston's Mills, N. J.	45.0	68	July 20, 1921
Shabakunk Creek, near Trenton, N. J....	13.1	107	April 7, 1924

In this connection the spillway capacity of some of the more important dams in New Jersey, as given in Table 3, may be of interest.

The Oradell Dam is the most important structure in New Jersey equipped with sluice-gates for passing flood waters. As the author points out, however, this was necessary in order to hold down the water level in the reservoir and

prevent the flooding of railroad and other unowned land. There are many dams in streams in the Coastal Plain of New Jersey that depend entirely on manual control of sluice-gates to pass flood waters. Most of these are structures built prior to State supervision. It is very desirable to eliminate so far as possible the necessity for operating gates to pass floods and to rely on overflow or other automatic spillways. In the Oradell Dam, under present conditions of operation as a water supply reservoir and with attendants constantly on duty at the pumping station immediately below the dam, it would seem that adequate provision has been made for gate control.

TABLE 3.—SPILLWAY CAPACITIES OF NEW JERSEY DAMS.

Name and location of dam.	Drainage area, in square miles.	Spillway capacity, in second-feet per square mile.	Remarks.
Pompton Lakes Dam, on Ramapo River...	160	70	Entire dam masonry
Boonton Dam, on Rockaway River.....	122	92	Entire dam masonry
Oradell Dam, Hackensack River.....	114	110	With sluice-gates
Wanaque Dam, Wanaque River.....	91	192	Earth dikes
Tintern Manor Reservoir, on Swimming River.....	48	190	Earth dikes
Oak Ridge Dam, on Pequannock River.....	27.3	180	Earth dikes

In some structures where inadequate spillway capacity is provided and it becomes the writer's duty to require enlargement and often added construction cost, he is met with the argument that the builder is not justified in making the increased expenditure to take care of a condition which may occur only once in a lifetime. Engineers use factors of safety of as much as six or more in many of their designs and yet the hydraulic engineer is often limited to a factor of two in many parts of his design. The writer is very strongly of the opinion that an endeavor to save money by providing a spillway of other than liberal capacity is poor economy for the owner as well as for the general public.

A. T. RICKETTS,* M. AM. SOC. C. E.—The wedge-piles one of which is shown in Fig. 8† were made to meet a difficulty experienced in driving the sheet-piling due to a tendency of the piles to drive out of plumb. The tops, which were held in perfect alignment by waling strips, traveled ahead of the lower ends. After 50 or 75 ft. of piling had been driven the inclination of the piles was sufficient to cause lateral vibrations under the hammer. An attempt was made to correct the difficulty by beveling the points so as to spread the joints at the bottom, but this was not successful.

No difficulty was experienced in driving the wedge-piles with their broad ends down. The angle of the wedge was made slightly greater than the inclination of the last straight pile driven so that the top of the pile following the wedge-pile inclined slightly toward the piles in place. Two or three of the wedge-piles were used under the spillway section.

* Associate Engr., Nicholas S. Hill, Jr., New York, N. Y.

† *Proceedings*, Am. Soc. C. E., October, 1925, Papers and Discussions, p. 1580.

MOMENTS IN RESTRAINED AND CONTINUOUS BEAMS BY THE METHOD OF CONJUGATE POINTS

Discussion*

By Messrs. Charles S. Whitney, Edward Godfrey, Felix H. Spitzer, and C. A. P. Turner

Charles S. Whitney,† M. Am. Soc. C. E. (by letter).‡—The authors have presented a very interesting graphical method for the analysis of continuous beams. As it has been proved that more economical reinforced concrete structures can be produced by designing them as elastic frames, any such method which simplifies the design is valuable. The writer has been using a very simple general analytical method§ for locating the conjugate points which should also be of interest. Fundamentally, this method is probably the same as that of the authors, but its derivation, appearance, and use are different. It is a general method for the solution of continuous frames composed of straight members the ends of which do not move horizontally or vertically. The derivation is effected by considering a single member as a beam with elastic supports. The notation used is shown in the diagrams, Figs. 39 to 44, inclusive.

The beam, $A\,B$, is considered as elastically restrained in such a manner that a unit moment at the left support will produce a rotation of that support equal to ϵ_a and a unit moment at the right support will produce a rotation equal to ϵ_b. The total rotation of the left and right supports, therefore, will be $M_a\,\epsilon_a$ and $M_b\,\epsilon_b$, respectively.

Considering the beam as a simple span without restraint, the superimposed loading will produce an angular displacement of the left end equal to α_{ao} and of the right end equal to α_{bo}.

Still considering the beam as simply supported, the angular displacement at the left and right ends produced by a unit value of M_a are, respectively, α_a and β. The same values when $M_b = 1$ will be called β and α_b. By Maxwell's theorem of reciprocal displacements, the values of β are the same in both cases. If the beam has a uniform moment of inertia, the values of α_a and α_b will be equal.

Using these displacements, the location of the conjugate points on the closing line of the bending moment diagram may easily be derived. Equating the angular displacements at the left support,

$$-M_a\,\varepsilon_a = \alpha_{ao} + M_a\,\alpha_a + M_b\,\beta$$

At the right support,

$$-M_b\,\varepsilon_b = \alpha_{bo} + M_b\,\alpha_b + M_a\,\beta$$

* Discussion of the paper by L. H. Nishkian and D. B. Steinman, Members, Am. Soc. C. E., continued from December, 1925, *Proceedings*.

† Cons. Engr., Milwaukee, Wis.

‡ Received by the Secretary, November 5, 1925.

§ See "Neuere Methoden zur Statik der Rahmentragwerke und der elastischen Bogentraeger," by A. Strassner, and "Methode der Festpunkte," by E. Suter.

or,

$$\left.\begin{aligned} M_a\,(\alpha_a + \varepsilon_a) + M_b\,\beta &= -\alpha_{ao} \\ M_b\,(\alpha_b + \varepsilon_b) + M_a\,\beta &= -\alpha_{bo} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (20)$$

Assuming the beam as shown in Fig. 40 with the actual elastic support at the left end and a simple support at the right end, apply a unit moment, $M_b = 1$, at the right end, and determine the location of the point of inflection. This point is called the "fixed point" and lies in the same vertical line as the conjugate point.

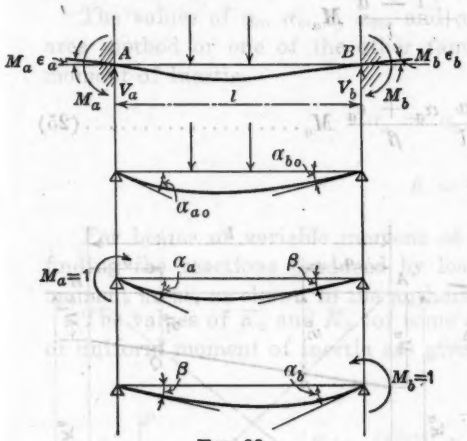

FIG. 39.

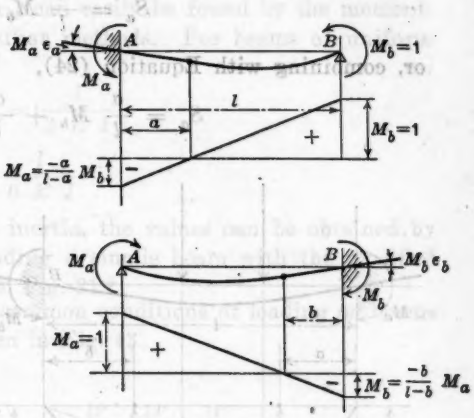

FIG. 40.

From Equation (20),

$$M_a\,(\alpha_a + \varepsilon_a) + M_b\,\beta = -\alpha_{ao}$$

In this case, there is no superimposed load and $\alpha_{ao} = 0$. Hence,

$$M_a\,(\alpha_a + \varepsilon_a) = -M_b\,\beta$$

Placing,

$$M_a = \frac{-a}{l-a}\,M_b \quad \text{(see Fig. 40)}$$

$$\frac{a}{l-a}\,M_b\,(\alpha_a + \varepsilon_a) = M_b\,\beta$$

From which,

$$a = \frac{\beta\,l}{\alpha_a + \beta + \varepsilon_a} \quad \ldots\ldots\ldots\ldots\ldots\ldots (21)$$

Similarly,

$$b = \frac{\beta\,l}{\alpha_b + \beta + \varepsilon_b} \quad \ldots\ldots\ldots\ldots\ldots\ldots (22)$$

These values of a and b are the abscissas of the conjugate points. For unrestrained beams, the values of ϵ_a and ϵ_b are infinite and a and b are zero. For rigidly fixed ends, ϵ_a and ϵ_b are zero and

$$\left.\begin{aligned} a &= \frac{\beta\,l}{\alpha_a + \beta} \\ b &= \frac{\beta\,l}{\alpha_b + \beta} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (23)$$

For prismatic beams, $\alpha_a = 2\,\beta$ and $a = b = \frac{l}{3}$.

The ordinates of the conjugate points will now be found. From Equation (21),

$$\frac{l-a}{l} = \frac{a}{l}\,\frac{\alpha_a + \varepsilon_a}{\beta} \qquad (24)$$

From the geometry of Fig. 41,

$$S_a = \frac{a}{l}\,M_b + \frac{l-a}{l}\,M_a$$

or, combining with Equation (24),

$$S_a = \frac{a}{l}\,M_b + \frac{a}{l}\,\frac{\alpha_a + \varepsilon_a}{\beta}\,M_a \qquad (25)$$

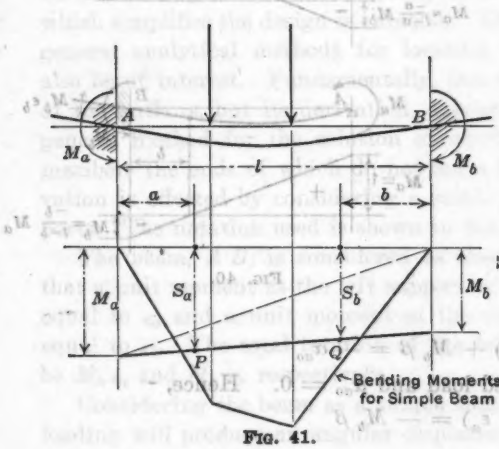

FIG. 41.

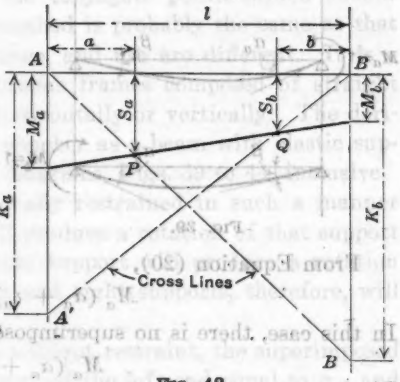

FIG. 42.

From Equation (20),

$$M_a = -\,M_b\,\frac{\beta}{\alpha_a + \varepsilon_a} - \frac{\alpha_{ao}}{\alpha_a + \varepsilon_a}$$

Combining with Equation (25),

$$S_a = -\,\frac{a}{l}\,\frac{\alpha_{ao}}{\beta} \qquad (26)$$

Similarly,

$$S_b = -\,\frac{b}{l}\,\frac{\alpha_{bo}}{\beta} \qquad (27)$$

These are the ordinates of the conjugate points, P and Q.

The points, P and Q, lie on the "cross lines", $A\,B'$ and $B\,A'$, which are useful and can be located as follows. The intercept, $A\,A'$, called K_a, is seen in Fig. 42 to be equal to,

$$K_a = \frac{l}{b}\,S_b = -\,\frac{\alpha_{bo}}{\beta} \qquad (28)$$

And, similarly,

$$K_b = -\frac{\alpha_{ao}}{\beta} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (29)$$

These cross lines are the loci of the conjugate points and depend for their location on the deformation of the unrestrained beam under the assumed loading. They are not affected therefore by the actual end conditions. The abscissas, a and b, are fixed by the end conditions and the elastic properties of the beam itself, but are independent of the actual loading.

The values of α_a, α_b, β, α_{ao}, and α_{bo} can easily be found by the moment-area method or one of the other familiar methods. For beams of uniform moment of inertia,

$$\alpha_a = \alpha_b = \frac{l}{3\,E\,I}$$

$$\beta = \frac{l}{6\,E\,I}$$

For beams of variable moment of inertia, the values can be obtained by finding the reactions produced by loading a simple beam with the modified moment areas, as shown in the authors' Fig. 21.*

The values of K_a and K_b for some common conditions of loading on beams of uniform moment of inertia are given in Fig. 43.

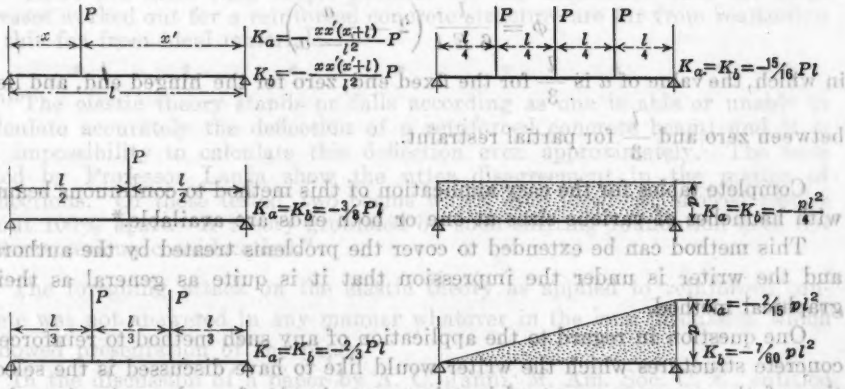

FIG. 43.

The value of ϵ_a is the angular displacement of the left support when $M_a = 1$. If the span under consideration is continuous with another beam to the left, ϵ_a will be equal to the angular displacement of the right end of this beam when $M_a = 1$. (See Fig. 44.)

$$\epsilon_a = \phi = \alpha_b' - \frac{a'}{l'}\beta'$$

the values of α_b', a', l', and β' being the values for the span to the left of the support being considered.

* *Proceedings*, Am. Soc. C. E., October, 1925, Papers and Discussions, p. 1613.

If there is more than one member framing into the left end of the loaded beam, the value of ϵ_a is determined by the expression:

$$\frac{1}{\epsilon_a} = \frac{1}{\phi_1} + \frac{1}{\phi_2} + \frac{1}{\phi_3} + \ldots + \frac{1}{\phi_n}$$

in which, ϕ_1, ϕ_2, etc., are the angular displacements for each member separately when acted on by a unit moment at the end. When there is a series of spans, the values of ϵ_a and a are determined successively, starting with the the left-end span. The values of ϵ_b and b are determined in the same way from the left-end span, in which case,

$$\epsilon_b = \phi = \alpha_a'' - \frac{b''}{l'' - b''}\beta''$$

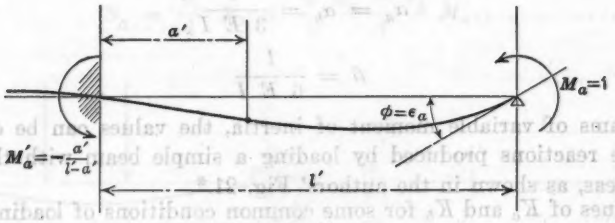

FIG. 44.

For members with a uniform moment of inertia,

$$\phi = \frac{l}{6\,E\,I}\left(2 - \frac{a}{l-a}\right)$$

in which, the value of a is $\frac{l}{3}$ for the fixed end, zero for the hinged end, and lies between zero and $\frac{l}{3}$ for partial restraint.

Complete tables for the easy application of this method to continuous beams with haunches of various sizes at one or both ends are available.*

This method can be extended to cover the problems treated by the authors, and the writer is under the impression that it is quite as general as their graphical method.

One question in regard to the application of any such method to reinforced concrete structures which the writer would like to have discussed is the selection of the proper moment of inertia to use in the formulas. As the solutions depend only on the relative moment of inertia of the different members, it has been suggested that the moments of inertia of the full concrete section be used, neglecting the steel entirely. The writer believes that this will ordinarily give good results in the case of continuous beams, but it would appear that some modification would be required if some members were very much more heavily reinforced than others. In continuous T-beam construction, it is customary to use the moment of inertia of the full T-section for those parts of the beams subjected to negative bending as well as those with positive bending.

* "Neuere Methoden zur Statik der Rahmentragwerke und der elastischen Bogentraeger" and "Methode der Festpunkte."

EDWARD GODFREY,* M. AM. SOC. C. E. (by letter).†—This paper, as the "Introduction" sets forth, was written primarily to be of use to designers in reinforced concrete construction. A secondary use is mentioned, namely, in computing secondary stresses.

On several occasions the writer has emphasized the fact that concrete construction, as proved by tests, does not act in a true elastic manner nor even approximately so. He has also stated that there are many considerations, apart from the one stated by the authors, namely, tedious methods, which make it superfluous to consider secondary stresses.

In the writer's paper entitled "Some Mooted Questions in Reinforced Concrete Design,"‡ the following statements were made:

"Professor Gaetano Lanza has given some data on recorded deflections of reinforced concrete beams.§ He has also worked out the theoretical deflections on various assumptions. An attempt to reconcile the observed deflections with one of the several methods of calculating stresses led him to the conclusion that:

"'The observations made thus far are not sufficient to furnish the means for determining the actual distribution of the stresses, and hence for the deduction of reliable formulae for the computation of the direct stresses, shearing stresses, diagonal stresses, deflections, position of the neutral axis, etc., under a given load.'

* * * * * * *

"There are more than ample grounds for the contention that the ideal stresses worked out for a reinforced concrete structure are far from realization in this far from ideal material.

* * * * * * *

"The elastic theory stands or falls according as one is able or unable to calculate accurately the deflection of a reinforced concrete beam; and it is an impossibility to calculate this deflection even approximately. The tests cited by Professor Lanza show the utter disagreement in the matter of deflections. Of those tested, two beams which were identical, showed results about 100% apart. A theory grounded on such shifting foundation does not deserve serious consideration."

The foregoing attack on the elastic theory as applied to reinforced concrete was not answered in any manner whatever in the long discussion which followed presentation of the paper.

In the discussion of a paper by A. C. Janni, M. Am. Soc. C. E., entitled "The Design of a Multiple-Arch System and Permissible Simplifications",‖ the writer made the following statements:¶

"The modulus of elasticity of concrete varies considerably, even for specimens made of apparently the same kind of materials. It also varies for different intensities of stress, which means a variation in the same cross-section of a member in bending. The first of these inconsistencies could not

* Structural Engr., Pittsburgh, Pa.

† Received by the Secretary, November 9, 1925.

‡ *Transactions*, Am. Soc. C. E., Vol. LXX (1910), p. 54.

§ "Stresses in Reinforced Concrete Beams," *Journal*, Am. Soc. Mech. Engrs., Mid-October, 1909.

‖ *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1142.

¶ *Loc. cit.*, p. 1189.

be allowed for mathematically; the other, if the law of its change should be discovered, might yield to analysis by adding manyfold to the complexity of an already overburdened mathematical solution.

* * * * * * *

"Furthermore, steel reinforcement in a concrete arch contributes materially to the lack of true elasticity and the fact that it is unstressed in its original condition. Shrinkage in the concrete puts initial stress in the steel, and any calculations of equivalent moment of inertia based on unstressed steel and concrete are vitiated by this undisputed fact.

"The whole fabric of the method of designing reinforced concrete arches as elastic structures stands or falls according as the deflection of a reinforced concrete beam under test can or cannot be accurately determined by computation. Tests prove that this deflection cannot even be approximated by computation. Here, in a nutshell, is the proof that the elastic treatment of the reinforced concrete arch is improper."

So far as the writer knows these arguments have not been refuted.

Reinforced concrete arches designed on the elastic theory and continuous beams are on the same basis and what is said concerning the applicability of the theory to one has equal force as to the other. If neither is an elastic structure, the elastic theory does not have application.

A structure to be elastic in the mathematical sense must not only exhibit what is commonly understood as elasticity, but it must have a constant or nearly constant modulus of elasticity in all its parts. The theory of elasticity, particularly such phases as apply to elastic arches and continuous beams, can have no application unless the modulus of elasticity is a constant. When two specimens of concrete made in an identical manner show variations of 100% in deflections, this means that the modulus of elasticity in these two specimens varies by that amount or more; or else, what is still more significant, it indicates that that modulus may be different for different parts of the same member. That difference would have to be more than 100% in order to make the averages to differ by 100 per cent.

Many experiments demonstrate the fact that the modulus of elasticity of concrete is quite uncertain and varies even to the extent of several hundred per cent. Besides, many tests indicate that there is a variation for different intensities of stress. The application of the elastic theory to a material such as this is a violation of the fundamentals of engineering mechanics.

So far as the writer knows no attempt has ever been made to reply to this objection to the elastic theory as applied to concrete and reinforced concrete. It is to be hoped that the authors will make serious reply to this objection. It is just as clearly a matter of exact analysis as any part of any solution of the problem of elastic concrete structures, and unless it can be controverted, every part of every such solution—a large total in engineering literature—is merely theoretical matter of no value.

On secondary stresses, with which the paper also deals, very complex mathematics has been introduced into the literature of engineering; and it is seriously questioned if any of this is of practical value. This highly theoretical matter has more than once been challenged as inappropriate and useless by the writer, chiefly on the ground that the malleability of steel vitiates all assumptions, and tests do not indicate any agreement between

the theory and fact, because the theory referred to ignores malleability in steel.

In a seven-page discussion* of a paper on "Secondary Stresses in Bridges"† by Cecil Vivian von Abo, Jun. Am. Soc. C. E., the writer set forth reasons why it is unnecessary to consider secondary stresses in steel work. Mr. von Abo's answer‡ to this is, "Mr. Godfrey questions the definition of 'secondary stress'".

Another paper§ further emphasizes the futility of the consideration of secondary stresses. A paragraph from that paper is pertinent:

"D. B. Steinman in his conclusions on the tests of the Hell Gate Arch says in *Transactions*, Am. Soc. C. E., Vol. LXXXII, p. 1071, that during erection, 'except for the smallest secondary stresses, the measured values were consistently lower than the calculatel values. For the higher percentages of secondary stresses the measured values are only a small fraction of the calculated values.' Also, 'the actual secondary stresses will generally be lower than the calculated values. There is an automatic re-adjustment of strains within a structure in such direction as to relieve the secondary stresses.' There can be no doubt whatever that this 'automatic re-adjustment' is simply the working out of the effect of toughness or malleability of steel, a property absolutely ignored by the theorist."

If, in the most elaborate set of tests ever made to investigate secondary stresses, so striking a disagreement with theory as that indicated in the last paragraph was manifest, is it not legitimate to ask, "Why should consideration of secondary stresses have any place either in engineering design or literature?".

FELIX H. SPITZER,‖ Assoc. M. AM. SOC. C. E. (by letter).¶—The graphic methods developed in the paper are simple, short, and easily checked and will soon replace the formulas with arbitrary coefficients in the design of continuous beams.

The writer has been familiar with the method previous to its publication through the courtesy of Mr. Nishkian and has used it in the design of several buildings, saving in the quantities of concrete and steel in the frame work as much as 20 per cent.

The designs resulting from this method call usually for larger negative moments and smaller positive moments than would have resulted from the use of building ordinance formulas. There takes place, to a certain extent, a shifting of the quantities from the center to the support. The negative or top steel becomes more important and its location has to be more insured. The general reduction of quantities cuts down the margin of carelessness and poor work.

For these reasons the writer recommends that the use of these graphic methods shall automatically call for better engineering field inspection than is given to present-day concrete building construction.

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* *Proceedings*, Am. Soc. C. E., April, 1925, Papers and Discussions, p. 644.
† *Loc. cit.*, September, 1924, Papers and Discussions, p. 969.
‡ *Loc. cit.*, September, 1925, Papers and Discussions, p. 1459.
§ *The Canadian Engineer*, May 19, 1925, p. 495.
‖ Cons. Structural Engr., San Francisco, Calif.
¶ Received by the Secretary, November 14, 1925.

C. A. P. TURNER,* M. AM. SOC. C. E. (by letter).†—The authors follow the ordinary beam theory disregarding rhombic deformation. That the graphic solution outlined is simpler than available analytical methods, appears open to question. This might be true were only the methods of Clapeyron, as simplified by Merriman, available; but a series of three-moment equations of negative moments is more difficult to solve than a series of‡ two-moment equations of negative moments by which the problem may be attacked.

Half the sum of the moments over consecutive supports plus§ the moment at the center of the span, equals the moment ordinate for the given loads at the center of that span considered as simply supported. From a series of these equations, is obtained the odd-and-even rule for center moments between supports, for negative moments over the supports, and for the shears at the support. For a uniform load and equal spans, these values may be written without computation.

In the solution for concentrated loads or unequal spans, the two-moment equations of negative moments permit the deduction of additional three-moment equations of center moments, and also give the relation that the negative moment over any interior support equals the numerical sum of the adjacent positive center moments between supports. Thus, this‖ analytical method, as the writer views it, is simpler than the graphic.

Graphical analyses of continuous beams are in general based on Mohr's theorem. The general expression for the three-moment relation of Equation (1)¶ appears to have been first published by Charles E. Green in 1875.**

The symmetry of the summation equations of beam theory permit two analogous but divergent graphical methods—that of fixed points determining the false polygon of deflection, as followed by Eddy,†† and Church,‡‡ and that of characteristic points from which the closing line of the moment area may be found, as detailed by Fidler,§§ whose treatment follows almost identically the same lines as the authors' and appears simpler in detail. Hence, a comparison by the authors of this method with their work would be of interest.

The authors' premise disregarding rhombic distortion involves the assumption that a plane normal section before bending remains plane after bending and normal to the axis of the beam; it assumes that the sum of the horizontal compressions balances the sum of the horizontal tensions, which, although true of the simply supported beam, is untrue for the continuous beam.

The equalizing of the horizontal tensions by the horizontal compressions takes place through horizontal shear, which changes sign at the neutral plane. The sum of the shears above the neutral plane constitutes the reaction to the sum of the horizontal compressions in simple flexure.

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* Cons. Engr., Minneapolis, Minn

† Received by the Secretary, November 16, 1925.

‡ "Concrete Steel Construction", by Eddy and Turner, Second Edition.

§ Plus becomes minus in case of zero load on the span.

‖ "Elasticity and Strength of Materials", by Turner, Section II.

¶ *Proceedings*, Am. Soc. C. E., October, 1925, Papers and Discussions, p. 1595.

** "Graphical Analysis of Trusses", Van Nostrand.

†† "Researches in Graphical Statics", by Eddy, 1878.

‡‡ "Mechanics of Engineering", by Church, 1890.

§§ "Treatise on Bridge Construction", 1887.

Under Newton's third law, action and reaction must be equal and opposite, but college textbooks and university professors teach in substance that in the mechanics of flexure, while action and reaction must be equal, the center of action of the positive shears need not for equilibrium be opposite to the center of action of the compressions. In that way they erroneously locate the neutral plane at the center of gravity of the section along the length of the beam. This erroneous location of the neutral plane dates back 240 years to the investigation of Mariotte in which he assumed without proof, *on peut concevoir*, that there are as many fibers stretched as compressed.

Now, in order that the action of the compressions and the reaction of the shears may be opposite, the neutral plane must be depressed below the gravity axis toward the end of the simply supported beam a distance of nearly one-tenth the depth, following a curve of the same order as the slope curve for various loadings, whether central, uniform, or other.

In order that the material may obey Hooke's law there must be a shearing strain in proportion to the shearing force and the plane section instead of remaining plane and normal to the axis of the beam is rotated from it and assumes the form of a reverse third degree curve; since the shears are the summation of the moment stresses following a linear law the shear stress area is parabolic, bounded by a second degree curve, and consequently the summation of the shear areas which determines the deformation is a third degree curve.

Applying this reasoning to the restrained beam, if it were hinged at the points of inflection the neutral plane at the hinge of the suspended span would be nearly one-tenth the depth below the center of gravity of the section and the neutral plane for the cantilever portion would be one-tenth the depth above that center. Since the material is not separated, these diverse positions of the neutral plane, when hinged, are forced together by interference of the rotational shear strains represented by a couple of one-half the total intensity of the moment stresses having a lever arm of approximately one-fifth the depth of the beam. Hence, for the restrained steel beam under uniform load having a ratio of $\frac{L}{d} = 10$, the maximum tension would be only three-quarters over the supports and one-half at mid-span of that computed from the erroneous location of the neutral plane following the common theory of flexure.

The author's proposed theory for the attainment of economic results in beam design is thus lacking in that degree of accuracy which permits the greatest economy. In this respect it offers little improvement over building codes, which likewise disregard shear distortion.

The criticism of the ordinary beam theory, because it disregards Hooke's law and Newton's third law as well, is applicable to the entire theory of secondary stresses. The bending of a prismatic beam is determined predominantly by the elongations and compressions parallel to the axis. In the column, on the other hand, before the stage of incipient failure has been reached, elongations are approximately normal to the axis and compressions

parallel to it. Its change in shape is determined, therefore, predominantly by the very shear strains which are disregarded in the theory of secondary stress. This disregard of shear strain reaches its greatest absurdity in building codes and in the rules of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, for determining the resisting section of restrained and continuous beams under external shearing force.

The procedure is first to assume that shearing force produces no shear strain. Then because shear strain causes shear failure it is provided for by assuming a unit shear stress unrelated by proportion to the shear strain that causes failure—computing that shear stress on the basis of the external shear force divided by the cross-sectional area instead of calculating it from the accepted summation equations of beam theory.

If, on the side of an elastic prism, lines parallel to and lines normal to the axis are drawn so as to divide the surface into squares, and the prism is bent sharply, the squares at the center will be deformed into trapezoids. The squares at the end at mid-depth will assume a rhombic form. At the center under uniform load there is no external shear force. Yet a rotational deformation has taken place and the squares have been deformed into trapezoids in which both diagonals are shortened in the compression zone and both diagonals lengthened in the tension zone. Toward the end of the beam, where the rhombic deformation occurs, the diagonal sloping downward toward mid-span from the end of the beam is lengthened and the other diagonal is shortened in both tension and compression zones. These two distinctive deformations have different laws of summation and arise from divergent causes.

Trapezoidal deformation is produced by the external shear force and is proportional to the moment at any point in the beam. Rhombic deformation at the neutral plane on the other hand is proportional to the horizontal shear stress (or the sum of the moment areas) or to the slope of the beam at the point in question. Cracking of the concrete occurs where the sum of these two rotational deformations is a maximum. Hence, shear failure occurs in the simply supported deep concrete beam toward the end; in the shallow beam toward the center.

Since, in the stress mechanism of a concrete beam reinforced in the bottom, the steel resists the direct tension, checking or cracking of the concrete occurs in the shallow beam because of the sharpness of curvature requiring a limitation in the percentage of steel. In the deep beam cracking is caused predominantly by rhombic shear strain.

In a given beam for the same kind of loading, with the same percentage of steel, and the same working stress on the steel, the rhombic shear strain in the simple beam is proportional to the external shear force for a given distribution but not for different distributions of loading. In the continuous beam, on the contrary, because it is made up of cantilever and suspended span portions, the rhombic deformation is zero at the support instead of a maximum as in the simple beam and is a maximum at the point of contraflexure. Trapezoidal deformation, on the contrary, is a maximum at the support and at mid-span for symmetrical loading.

From these divergent laws of summation there results a resistance to external shear force which may be three to four times as great in the continuous beam per unit of section as that developed in the simple beam, yet building code laws and the rules of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, still err in requiring provision for shear strain in the continuous beam as though it were comparable in its mode of resistance to the simple beam.

It is to be regretted that the authors did not carry out their investigations on a stricter application of Hooke's law so that their theory would accord more closely to the conditions met in practical work.

The divergence from exact theory by the disregard of shear strain may be summarized as follows: The neutral plane in the simple beam passes through the center of gravity of the section only at the locus of maximum deflection. It dips downward toward the end of the beam in a curve of the same order as the slope curve. In the continuous beam, on the contrary, due to the interference of shear distortion, the neutral plane passes through the center of gravity of the section at the points of contraflexure only. It rises above the center of gravity at the supports and drops below it between supports, following in like manner a curve of the same order as the slope curve for the given applied loads. Shear distortion increases the deflection of the simple beam but reduces the deflection of the continuous beam just as in the case of the statically indeterminate arrangement of a load on two springs of different rigidities. The deflection is less for the indeterminate arrangement than for any statically determinate arrangement. In the statically indeterminate restrained or continuous beam the deflection is reduced not only by the divergent distribution of moment magnitudes but by interference of shear distortions as well.

It appears that heretofore no general definition has been formulated covering jointly what may be called the neutral plane in the homogeneous beam and in the reinforced concrete beam as well. The writer would define this plane as the locus of consecutive points of right shearing stress in vertical planes parallel to the axis where tension, compression, and shear have the same magnitude. This state of right shearing stress in the composite beam differs from that in the homogeneous beam in that as the shears are equalized at the steel instead of at the neutral plane the states of right shearing stress at the neutral plane are rotated one-half an octant from that of the homogeneous beam, in keeping with the depression of the plane of equalization of the shears one-half the depth below the neutral plane. This state of facts indicates the error of the supposition that tension in the concrete offers material resistance to flexure and that the divergence in the newly cast beam between steel moment times the lever arm and the applied moments of the load must be attributed to the reaction of the residual stresses induced during the hardening of the concrete about the steel.

It is to be noted that the proportional reduction of the deflection caused by interference of shear distortion decreases as the square of $\frac{d}{L}$ decreases

and in like manner does its influence in reducing the intensity of the tension over the support at the top and between the supports at the bottom.

The much greater rigidity of a column in flexure when loaded with the weight of several stories on the one hand as compared with the same column carrying the relatively light load of the deck structure on the other is one of the surprising observations in testing concrete buildings. Here the ordinary beam theory as applied in the paper gives no indication of the phenomena, but it may be explained by the variation of the relative flexural resistance resulting from divergent stress areas at critical sections of the column.*

The bearings of the continuous beams in a concrete building are more or less fully restrained either by the rigidity of the columns into which they frame or by the torsional rigidity of the carrying beams and slabs through which continuous joists may extend. The three-moment relations thus become indeterminate, as the number of equations of condition is one less than the number of unknown moments over the supports, forcing the practical designer to use other relations in solving his problem.

The method of the authors is therefore applicable to continuous steel beams with hinged supports rather than to ordinary reinforced concrete construction. As the summation equations relating to trapezoidal and rhombic deformation follow divergent laws, it is simpler to treat each kind of deformation separately, determining the resultant state of stress by combining the two. The authors' analysis forms a correct basis for the preliminary computation of trapezoidal deformation. The resultant unit stress in tension is modified by combining the trapezoidal and rhombic deformations.† In the foregoing discussion it is to be kept in mind that the center moment is not the maximum positive moment but the positive moment at the center of span, which may be utilized as a kind of relation from which the maximum negative and positive moments may be determined.

* "Elasticity and Strength of Materials," Section IV, p. 8.

† *Loc. cit.*, Section II.

RELATION OF THE OHIO RIVER AND ITS TRIBUTARIES TO TRANSPORTATION IN THE UNITED STATES

A SYMPOSIUM

Discussion*

BY MESSRS. M. G. BARNES AND C. I. GRIMM

M. G. BARNES,† M. AM. SOC. C. E. (by letter).‡—The writer has been very much interested in Mr. Alfred's paper.§ Through a series of mathematical exercises the author has arrived at the conclusion that the ultimate cost of haul *via* the Ohio River is 40% in excess of the haul by rail, and that "even this figure is not comparable to the railroad cost per ton-mile, as a ton-mile by river is not the equivalent of a ton-mile by rail."

The conclusion is so opposed to the writer's conclusions as to the relative cost of rail and water haul, that he was impelled to make a more detailed analysis of the paper. He did not think that railroads could compete with transportation by water over such an important river as the Ohio. Mr. Alfred's train of thought reminds one of the argument pursued by the Bureau of Railway Economics in its consecutive *Bulletin No. 293,* in which the Bureau attempted to prove that the cost of transportation *via* the New York State Barge Canal was greater than the cost of transportation by rail. The argument made very interesting, if true, reading, but having reached such astounding conclusions the writer was in that case also impelled to investigate more closely. In checking the Bureau's arithmetic he found that it had quite conveniently misplaced the decimal point in computing the cost of haul by water, which resulted in finding "cost of operation, maintenance, and repair" ten times too great. When this correction was made, and using the Bureau's own argument, it was shown that the cost of haul by water was less than the cost of haul by rail, as one must assume would be the result when the figures are properly applied. Has not Mr. Alfred fallen into a similar error in his train of thought?

The author is challenging the economy of transportation by the Ohio River from Pittsburgh to Cairo as a unit, whereas he is using shipping statistics that apply almost entirely to local freight shipping over a reach of 358 miles of river, which he states was all that was completed in 1923, the year in which his shipping statistics were compiled. Through shipment on the Ohio River

* Discussion continued from December, 1925, *Proceedings.*

† Chf. Engr., Div. of Waterways, State of Illinois, Chicago, Ill.

‡ Received by the Secretary, November 19, 1925.

§ *Proceedings,* Am. Soc. C. E., October, 1925, Papers and Discussions, p. 1663.

over an improved waterway such as the Government is now in the process of constructing, has not yet been possible. Several of the locks and dams are yet incompleted. Manifestly, it is not fair to use the present local shipments as an argument to show the wisdom or fallacy of improving the entire length of the Ohio River. Moreover, the Ohio River is only one unit in a system of waterways, as is one line of a railroad a unit in a railroad system, such as the Pere Marquette, for example. Obviously, it would be unfair to select some part of the Pere Marquette Railroad which might be operated at a serious loss, and argue from that that the entire railroad system is a failure.

The writer has compiled Table 16 as a modification of Mr. Alfred's Table 13*, to show comparison between railroad shipments and water shipments over the Ohio River System. The data in the second column of Table 13 are derived from Tables 3 and 4† of the paper by Colonel Kutz.

TABLE 16.—WATER AND RAIL SHIPMENTS ALONG THE OHIO RIVER.

	Railroads in United States.	Ohio River System, 1924.	Percentage of railroad.
Length, in miles	250 183	2 471	0.99
Tons of freight	2 333 000 000	38 071 601	1.63
Tons of freight per mile of length	9 065	15 400	170.00
Ton-miles	416 211 000 000	2 105 886 000	0.51
Ton-miles per mile of length	1 663 228	852 220	51.25
Cost, including equipment	$22 681 000 000	.....	.....
Cost per mile, including equipment	$90 658	.....	.....
Estimated cost of improvement	.....	142 161 023	.....
Estimated cost per mile of improvement	.....	57 532	.....
Balance for floating equipment and terminals to make cost equal to cost of railroads per mile	.....	33 126	.....
Total for Ohio System equipment and terminals	.....	81 854 346	.....

A similar comparison with the completed portion of the Ohio River tonnage shows that the tons of freight per mile of length is twice as much as on the railroads of the United States.

It will be seen from Table 16 that the length of the improved waterways of the Ohio River System is practically 1% of the length of the railroads of the United States, whereas the freight shipments by the Ohio River System were 1.63% of rail shipments in the United States. Also, that the shipment per mile of length on the Ohio River System was 70.0% greater than the average shipment by rail over the railroads of the United States. This is not such a bad showing for a transportation system yet incomplete and unusable in many places except for local business. The average haul is short because of the fact that through shipments cannot be made over the improved waterways.

The estimated cost per mile of improvement on this river system is $57 532, whereas the cost per mile, including equipment, on the railroads of the United States, is shown by Mr. Alfred to be $90 658. This leaves a balance of $33 126 per mile of waterways in the Ohio River System to be used for floating equipment or terminals, or for the total length of the system of 2 471 miles, the

* *Proceedings*, Am. Soc. C. E., October, 1925, Papers and Discussions, p. 1666.
† *Loc. cit.*, p. 1645.

handsome sum of $81 854 346 as available for floating equipment and terminals for the Ohio River System before the total sum has reached the average total cost, including equipment, of the railroads of the United States. It will be seen that the amount available for the floating equipment and real estate on the Ohio River System is surely adequate to take care of a very substantial business.

Mr. Alfred has computed* the items of interest, depreciation, taxes, and maintenance on the Ohio River as $9 656 000 annually, and that "this amounts to a cost of 1.29 cents per ton-mile based on the volume of traffic for 1923." Here, again, the writer wishes to invite attention to the fact that at this time only 358 miles of the river had been completed, and that no through traffic over the improved waterway was possible.

The author states further "as the volume of traffic decreases, this cost will increase." He fails to state the converse of this—that as the traffic increases, the cost will decrease. Although the writer does not endorse the findings of Mr. Alfred in these four items he will take them at their face value and compute the value of the Ohio River improvement on this basis. By using the argument of H. G. Moulton, M. Am. Soc. C. E., Mr. Alfred arrives at a present cost of haul for coal over the Ohio River from Pittsburgh to Cairo of 2.7 mills per ton-mile. Adding about 10% to this for the shorter haul will give a shipping cost by barge of 3 mills per ton-mile, which is equal to the transportation cost on the Monongahela River over a distance of 42 miles, as reported by Colonel Kutz, who also states* that the rail charge for shipping coal over the same distance is 19 mills per ton-mile, or a difference of 16 mills per ton-mile in favor of transportation by water. Further, Colonel Kutz submits a chart (Fig. 1†) showing the cost of haul of coal by rail for distances up to about 600 miles. In Table 12‡ Mr. Alfred finds that the average water haul for coal and coke in 1923 on the Ohio River was 83 miles. From Colonel Kutz's table for this distance is found the rail charge of about 16½ mills, or, say, 16 mills. This would show a saving of 13 mills per ton-mile in favor of water haul.

In the case of sand, gravel, and stone shown in Mr. Alfred's Table 12, the haul averages only 13 miles, and the difference between the water and the rail haul over such a short distance is even greater than for the coal haul of 83 miles, but for the purposes of this discussion it will be assumed that the saving is the same. In the case of the metal and metal products, Mr. Alfred finds the average haul is 318 miles.

Colonel Kutz shows that for a distance of 311 miles the rail charge for fifth-class freight, which is the same as for steel products, is 18.6 mills per ton-mile. Mr. Alfred feels that the cost by water for these higher class commodities should be greater than the cost for transporting coal. The writer agrees with Mr. Alfred on this subject and suggests an increase of 20% over the coal charge. This would give a cost of haul by water of 3.6 mills per ton-mile for fifth-class freight, such as steel products. It is interesting to note that

* *Proceedings*, Am. Soc. C. E., October, 1925, Papers and Discussions, p. 1667.
† *Loc. cit.*, p. 1655.
‡ *Loc. cit.*, p. 1648.
§ *Loc. cit.*, p. 1665.

Mr. Theodore Brent, Federal Manager for the Mississippi-Warrior Service, states that the average revenue to that barge line is about 3½ mills per ton-mile. This checks very closely with the writer's figure.

Mr. Alfred argues that if these commodities were forwarded by rail the distance would be less than 318 miles. That is probably true, but, on the other hand, for distances less than 400 miles the cost of haul by rail per ton-mile increases at about the same rate as the distance decreases, and hence this difference in distance is negligible. (See Table 8* and Fig. 3† of the paper by Colonel Kutz.)

For example, in Table 14,‡ Mr. Alfred finds the average length of water haul to be 44% greater than the rail haul. This relation applied to the 318-mile water haul gives an equivalent rail haul of 221 miles. By interpolation from Fig. 3 of the paper by Colonel Kutz, the rail rate for 221 miles is 25 mills. The total rail charge then becomes $5.52, or 17 mills per ton-mile of water haul. In the same manner it may be shown that the rail rate per ton-mile of water haul for "all other commodities" is 19 mills. The saving to shippers for these two items is 13.5 and 15.5 mills per ton-mile. Inserting these figures in the totals given by Mr. Alfred in Table 12 gives Table 17.

TABLE 17.—ECONOMY OF OHIO RIVER TRANSPORTATION.

Commodities.	Tons.	Percentage of total.	Ton-miles.	Average haul, in miles.	Savings per ton-mile, in mills.	Total savings.
Coal and coke	6 014 708	73.0	503 315 936	83	13	$6 543 107
Sand, gravel, and stone	1 372 404	16.6	17 274 151	13	13	224 564
Metals and metal products	389 792	4.7	123 972 709	318	13.5	1 673 685
All other commodities	476 617	5.7	104 557 690	219	15.5	1 620 644
Total	8 280 521	100.0	749 120 486	90	....	$10 062 000

Table 17 shows that the total saving to shippers on the Ohio River alone for the year 1923 was $10 062 000, which exceeds the $9 656 000, overhead charges found by Mr. Alfred.

Be it remembered that this is on a river 968 miles in length only 358 miles of which had been completed and opened. This is not such a bad showing for a stream in the condition of the Ohio River in 1923. When the Ohio River improvement has been completed steel products can be forwarded on regular schedules from Pittsburgh to St. Louis, Memphis, New Orleans, and other points, and heavy commodities, such as lumber, sulphur, bauxite, sugar, coffee, canned goods, and similar articles, can be brought to Pittsburgh from down-stream points. A very material through commerce will be created, which will show even greater benefits than now exist over a waterway admittedly incomplete. Surely with this increased through traffic the shipping interests will realize the converse of Mr. Alfred's statement and may well expect that as the volume of traffic increases the fixed charges per ton will decrease.

* *Proceedings*, Am. Soc. C. E., October, 1925, Papers and Discussions, p. 1651.
† *Loc. cit.*, p. 1652.
‡ *Loc. cit.*, p. 1668.

C. I. GRIMM,* ASSOC. M. AM. SOC. C. E. (by letter).†—The paper by Colonel Kutz‡ presents in condensed form much valuable data that have not been readily available to engineers and permits making an intelligent analysis of a subject that has frequently been discussed from an inaccurate or biased standpoint. Both Colonel Kutz and Mr. Alfred§ have as a result of an analysis reached the same general conclusion, namely, that present savings in transportation costs on the Ohio River are less than fixed charges plus the cost of maintenance and operation. While Mr. Alfred infers that the former never will be as great as the latter, Colonel Kutz estimates that the savings are approximately 60% of the fixed and operating charges on the completed reach of the river, extending from Pittsburgh to Cincinnati. The writer whose duties include the making of detailed studies of this subject accepts this figure as reasonably correct, realizing that the available data do not permit of mathematical accuracy in such an estimate.

Mr. Alfred has based fixed and operating costs on a completed project, assuming that the traffic will never be greater, but probably less, than at present. If his view of the future is accepted success cannot be predicted for the project, but there are many reasons for assuming a far more optimistic attitude. A railroad in an incompleted state and working at the low ratio of capacity that now prevails on the Ohio River, could not make much of a showing in competition with completed and interconnected lines.

The traffic density on the Ohio River at present is less than 1 000 000 ton-miles per mile of river, while its ultimate capacity is easily ten times this amount. The principal coal roads in the Ohio Valley had a traffic density of 4 000 000 ton-miles per mile of main track in 1922, which was not a year of heavy traffic, and with this traffic density, the fixed charges plus maintenance of way, was 1.9 mills per ton-mile, or about 40% of the corresponding charges on that part of the Ohio extending from Pittsburgh to Cincinnati.

The Monongahela River has a traffic density of 20 000 000 ton-miles per mile on its lower 60 miles, while for the entire 125 miles of improved river, the traffic density is about one-half as great, the traffic on the upper half being very light. It seems obvious that the capacity of the Ohio River exceeds by far that of a double-track railroad, but as its future use will be governed by its ability to get traffic rather than by its capacity to carry this traffic, the latter is not an important economic consideration.

Some further statistics, more or less general, that may be of assistance toward forming a general view of the traffic possibilities of the river are, as follows.

Population.—The population of the Ohio Valley in 1920 was 15 500 000, or 76 per sq. mile as compared with 116 for Illinois, 142 for Ohio, and 195 for Pennsylvania. In these three States, there is 1 mile of railroad for every 650 population. The population of this basin amounts to 10 000 per mile of Ohio River and canalized tributaries. In 1920 the population of cities and

* Asst. Engr., U. S. Engr. Office, Cincinnati, Ohio.
† Received by the Secretary, November 27, 1925.
‡ *Proceedings*, Am. Soc. C. E., October, 1925, Papers and Discussions, p. 1642.
§ *Loc. cit.*, p. 1663.

towns on the banks of the Ohio was 2 150 000, or 2 200 per mile. The population of the Ohio Basin increased 10.7% from 1910 to 1920 while the cities and towns on the river showed an increase of 13.7%, as compared with 14.9% for the entire United States.

Potential Tonnage.—The tonnage per capita handled by the railroads of the United States has been increasing rapidly as indicated by the following statistics:

1880	6.5 tons
1890	11.1 "
1900	13.2 "
1910	19.6 "
1920	24.0 "

These figures include originating tonnage and that received from other lines. Statistics show that originating tonnage is about 55% of the total, or that it was about 13 tons per capita in 1920, 5 tons of which was coal. The originating tonnage other than coal on the Upper Ohio is particularly heavy on account of the great steel industry practically all the furnaces and mills of which are situated on the river. The annual capacity of the blast furnaces on the Ohio and its tributaries is approximately 20 000 000 tons, with an average production of 15 000 000 tons.

The basin of the Ohio River produces 55% of the 600 000 000 tons of bituminous coal that is mined annually in the United States and nearly 200 000 000 tons of this coal is shipped from mines situated in counties that border on the river or its canalized tributaries.

The originating tonnage in the Ohio Basin in 1920 may be roughly estimated as follows:

Coal	300 000 000 tons
Other tonnage (at 10 tons per capita)	155 000 000 "
Total	455 000 000 "

TABLE 18.—GROWTH IN RAILROAD TONNAGE IN THE UNITED STATES.

Year.	Tonnage.	Ratio of 1880.	Percentage of increase.
1880	320 000 000	1.	...
1890	700 000 000	2.18	118
1900	1 000 000 000	3.	43
1910	1 800 000 000	5.63	80
1920	2 500 000 000	7.8	39

The ton-mileage based on the average haul for the United States of 310 miles per originating ton is 141 050 000 000, less than 1.5 of which was handled by the rivers.

Growth in Tonnage.—The tonnage handled by the railroads of the United States has increased since 1880 as shown in Table 18.

The total increase has been equivalent to a steady rate of 66.7% each decade, while the rate since 1900 corresponds to 58% each decade. Since these figures do not include tonnage handled by waterways and highways, it is conservative to predict total future tonnage increases as 60% in each decade.

From the foregoing it appears that the tonnage in the Ohio Basin other than coal may be expected to increase about 100 000 000 tons from 1920 to 1930, while the tonnage of coal as correctly shown by Mr. Alfred will not increase greatly. There seems to be no doubt that the growth in this valley will demand extensive expansion of transportation facilities and that the Ohio River System if it provides economical transportation may easily absorb during the next twenty-five years a sufficient tonnage to justify the investment in locks, dams, terminals, and fleets, without depriving the railroads of the business they require for profitable operation, providing there is proper co-ordination between the railroads and this waterway system.

Coal has always been an important item in Ohio River tonnage as it is with the railroads in the Ohio Valley. The Ohio River tonnage from 1890 to date has been 60% coal. The 1924 tonnage was 53% coal and coke and 34% sand and gravel, with a similar percentage indicated by the 1925 statistics to date. The tonnage of the Chesapeake and Ohio, Norfolk and Western, and Hocking Valley Railroads was 80% products of mines (mostly coal) in 1922. The ton-mileage of coal, sand, and gravel handled on the Ohio is at present about 75% of the total, due to the fact that the average haul of coal, sand, and gravel is short. The average haul of coal on the railroads mentioned is long and their ton-mileage of coal will be at least 75% of the total.

The accompanying diagram (Fig. 6) shows, as far as records are available, the total Ohio River coal tonnage as well as the shipments from Pittsburgh to the lower river and shipments from Huntington, W. Va., and the Kanawha River. The former has disappeared, but the latter is now greater than at the date of opening the competing Chesapeake and Ohio Railroad in 1889, while as a result of developments in the steel industry other movements of coal have developed to replace the loss of the large tonnage of Pittsburgh coal moving to the lower river. The steel industry is extending down the river and its further development promises continued growth in tonnage, of river coal.

The rapid growth in the demand for electrical energy will result in a large expansion of steam power plants in this great coal region and these plants by reason of their water requirements must locate on large streams. As the cost of transporting coal from the mines to the load center of a power system is generally equal to, or less than, the cost of transmitting the power from a power plant at the mines to this load center, it is evident that steam plants having their load center near the Ohio River or to the north and west of it, and their coal supply in West Virginia or Kentucky, will ship this coal to power plants on the Ohio. This is actually indicated by the action of two large power plants recently constructed on the Ohio that will receive their coal by river.

The increased use of oil as fuel, while it tends to decrease the rate of growth in coal production increases the field of water transportation since this commodity is particularly adapted to river transportation by reason of safety,

low haulage costs, and low terminal costs. The oil traffic on the Ohio River has grown steadily during the past eight years, until it is now an important factor. The same may be said of iron and steel products, and it is estimated that from 20 to 25% of the present Ohio River ton-mileage is in these two commodities, traffic in which has developed almost entirely during this period.

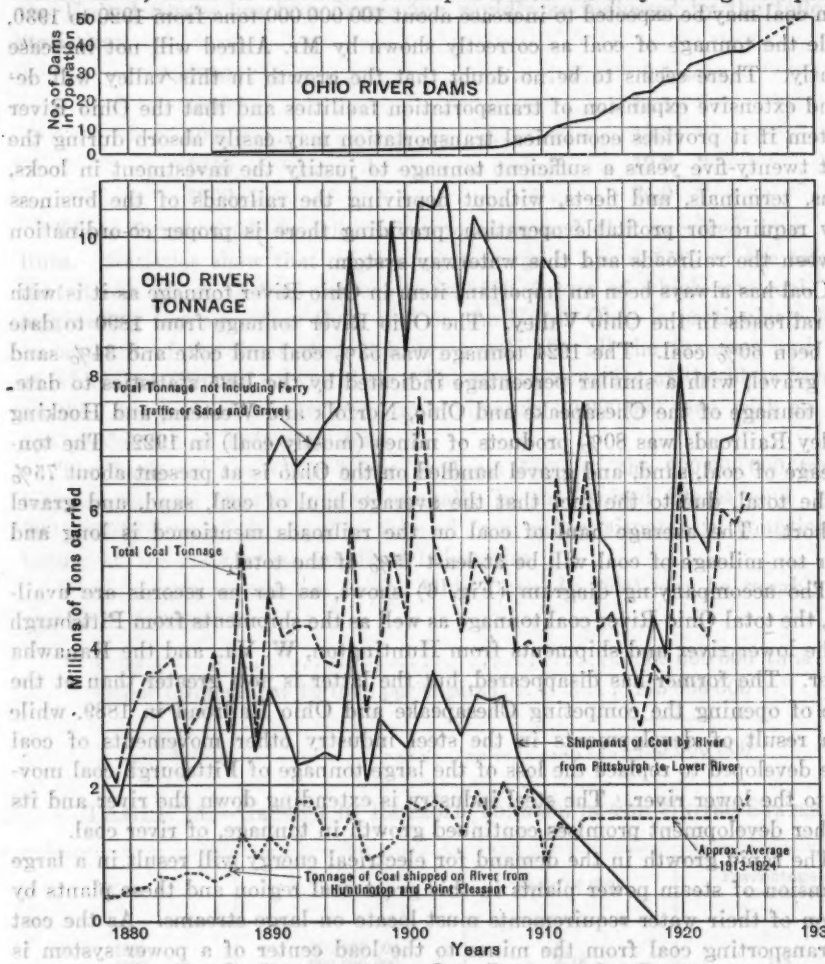

FIG. 6.—DEVELOPMENT OF OHIO RIVER AND ITS TRAFFIC.

It will be noted from Fig. 6 that there was a decline in Ohio River tonnage from 1905 to 1918 coincident with the decline in shipments of Pittsburgh coal to the lower river, while from 1918 to date there has been a general increase in tonnage coincident with the increase in tonnage of iron, steel, and oil, and partly composed of these commodities.

Colonel Kutz estimates (Table 4*) fixed charges on the completed project at 8.29 mills per ton-mile with 1924 traffic, while Mr. Alfred† based on the 1923

* *Proceedings*, Am. Soc. C. E., October, 1925, Papers and Discussions, p. 1645.
† *Loc. cit.*, p. 1667.

traffic estimates 12.9 mills. It is evident that traffic must increase so as to reduce the fixed charges well below these figures in order that the project may be justified.

From a former discussion by the writer,* are quoted (Table 19) data relative to coal-carrying railroads.

TABLE 19.—STATISTICS OF OHIO RIVER VALLEY RAILROADS.

	Pittsburgh and Lake Erie Railroad.	Buffalo and Lake Erie Railroad.	Hocking Valley Railway.	Norfolk and Western Railroad.	Chesapeake and Ohio Railroad.
Products of mines, percentage	64	90	83	80	79
Average haul, in miles	60	108	117	279	263
Average receipts, in mills per ton-mile	13.3	7.76	7.84	7.45	6.86
Average receipts per ton	$0.86	$0.84	$0.92	$2.08	$1.88
Interest on road plus maintenance of way in mills	2.31	1.58	1.96	1.82	1.60

To reduce fixed charges, plus cost of maintenance and operation on the river, to a figure as low as the corresponding charge on these roads, the river traffic must increase fivefold. This is, however, not a true criterion since the sum of other river transportation costs is less than those of railroads, as shown in the discussion referred to.

The average haul on the Ohio River is about 80 miles and the traffic as a whole is comparable with that of the short-haul (first three) railroads listed, but is practically all one-way, whereas these railroads have about 65% of their car-mileage in loaded cars. Taking these factors into consideration, it is estimated that the cost of handling all Ohio River traffic by rail would be $1.00 per ton, or 12.5 mills per ton-mile of river distance. The probable water cost is 5 mills per ton-mile, or $0.40 per ton and the terminal differential is estimated at $0.20 per ton, leaving a saving of $0.40 per ton as compared with rail shipments. On this basis, the tonnage on the completed project must increase to 20 000 000 or 25 000 000 tons to effect annual savings equal to annual costs carried by the Federal Government.

The foregoing estimate involving the use of averages is subject to many inaccuracies and a detailed estimate based on an analysis of the savings on each specific movement is the only method of obtaining more accurate results. As shown by Mr. Begien,† one of the important river hauls is a part of a longer through haul and this is true of many other river hauls. For such cases the estimate of savings is not simple, and results for specific movements may be widely different from those obtained by the methods of the preceding estimate. Data are not available for an analysis of all river movements, but a consideration of the major movements indicates that the foregoing estimate based on average conditions is reasonably correct.

As pointed out by Messrs. Alfred and Begien, dependability of service is a recognized advantage in a transportation facility and in this respect the railroads that compete with the Ohio have been far superior to the river in

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 571.

† *Proceedings*, Am. Soc. C. E., November, 1925, Papers and Discussions, p. 1883.

its uncanalized or partly canalized state. However, with the completion of the improvement, the railroads will possess no marked advantage in this respect. Navigation on the river will then be interrupted only by ice and extreme high water. Interruptions by the former will be greatest in the upper part of the river. At Lock No. 11, 76 miles from Pittsburgh, navigation has been suspended on account of ice an average of 7 days per year during the past 7 years, while high water has added 3 days more. On the Monongahela River during the past 25 years, ice and high water have interrupted navigation an average of 9 days per year. Velocities exceeding 3 miles per hour occur on the Ohio about 10% of the time in the steep upper end and about 5% of the time in the lower river, while velocities exceeding 2½ miles per hour prevail about 25% of the time. For an operation that is loaded down stream, towing conditions when the dams are lowered are no less favorable than those prevailing while the dams are up, until the river reaches a stage where the velocity is about 3½ miles per hour, but become less favorable as the velocity increases further and make towing impracticable when a velocity in excess of 4 miles per hour prevails.

The early growth of commerce on the Ohio River resulted primarily from its use in connection with marketing, which use gradually declined because of its inability in an uncanalized or partly canalized condition to meet railroad competition in this field. The recent increase in river tonnage has resulted primarily from its use as an agency of production. The former use resulted in long hauls of various products, while the latter involves shorter hauls of raw materials. It is recognized that extensive foreign and domestic markets to the south and southwest are open to the industries of the Ohio Valley and it is expected that the improved facilities offered by the river when it is completed will again build up its marketing commerce. A recent industrial survey report made by the Technical Advisory Corporation of New York, N. Y., for the City of Cincinnati, refers particularly to these markets for Cincinnati products and states as follows regarding the Ohio River: "The Ohio River improvement is to be completed in 1928. This simply gives Cincinnati an opportunity—nothing more".

The situation as viewed by the writer is summarized, as follows:

1.—The Ohio Valley with its great mineral and other resources is one of the richest in the world.

2.—Population and industry in this valley will keep pace in growth with that of the nation.

3.—Industry and population of the valley will center in the future, as they have in the past, on the Ohio River.

4.—The Ohio River where canalized offers an economical and dependable means of transportation.

5.—With equitable relations between it and the railroads which are recognized as the major transportation agency of the valley, this river will carry in the future a commerce that will justify the expenditures made on it by the Federal Government.

CO-ORDINATION OF IRRIGATION AND POWER

Discussion*

By Messrs. J. D. Galloway, Fred. H. Tibbetts, Barry Dibble, Markham Cheever, and M. C. Hinderlider

J. D. Galloway,† M. Am. Soc. C. E. (by letter).‡—This paper is a comprehensive and general review of the irrigation and power problems that exist on most rivers of the Western States. The diversity of the problems is illustrated by the contrast between physical conditions that exist on different rivers. In one case, as in the streams of the Sierra Nevadas, all power development is on the mountain section of the river and practically all irrigation on the broad plains below. In another case, such as the Deschutes River, or in the Green River Valley, all irrigation is above and the power development below. However, some general principles are applicable to all situations and it is of interest to discuss, in some detail, a few of the points mentioned in the paper.

The writer believes that too much emphasis has been placed on the apparent conflict between power and irrigation. This idea has been fostered by demagogues and politicians. The underlying cause has not been so much an inherent difference between the two uses of water as it has been that irrigation development, at least in recent years, has become a public function, while power development has been made by private corporations. Ever since the beginning of the hydro-electric industry, it has been the target for all forms of State and National regulation and restriction, beyond that of any other industry with the possible exception of the railroads. No real difference exists between the two agencies. Both are made up of groups of individuals organized under the laws of the State to perform certain necessary functions. It will be admitted that, as a matter of necessity, the production of food is more important than the generation of energy, but in any ordinary community, or in the United States as a whole, there is ample food, and all communities require power.

Under this conception, it is as important to develop power as it is to provide irrigation as a factor in the cycle of food production. Both are required in any community and it is a false principle to favor one as against the other, merely because of a difference in type of organization. The situation in Green River Basin, Wyoming, illustrates this point. When the Utah Power and Light Company applied for a license to develop power at the Flaming

* This discussion (of the paper by William Kelly, M. Am. Soc. C. E., presented at the Summer Meeting, Salt Lake City, Utah, July 8, 1925, and published in December, 1925, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr., San Francisco, Calif.

‡ Received by the Secretary, June 23, 1925.

Gorge site, it is to be presumed that additional power was required in the territory served by the Company. Yet Restriction *B* of the preliminary permit, as given by the author,* was practically a prohibition against building the power plant. No capital, private or public, should ever be put into power development for which the water supply may be progressively reduced by irrigation until the stream is nearly dried up. It would seem that under such conditions a balance should be established between the power requirements and those for irrigation, as both are necessary to a community. This point becomes self-evident when, as in many cases, much power is used for irrigation pumping or for drainage. To prohibit power development, as in the case cited, is to injure irrigation. It is to be hoped that some day the American people will become sane on this subject and repeal laws that definitely place any irrigation project, no matter how dubious its merits, above a very necessary power development.

In most cases there is no conflict between irrigation and power; both are mutually beneficial. The writer has already set forth this view at length.† Herewith, is reproduced Fig. 17,‡ a hydrograph of a stream on which is superimposed an irrigation demand curve and a power demand curve. This form is applicable to most rivers where power is developed in the mountains and irrigation on the plains below. To the extent that reservoirs for power are installed, there is assistance to irrigation during the irrigation season of any one year. In addition, if there is an irrigation reservoir on the lower reaches of the stream, water discharged from power system reservoirs in the period after the irrigation season is caught by the irrigation reservoir and retained for the next year's irrigation. The regulation of the stream over more than one year is thus accomplished. Where, as in California, the annual variation in total discharge may range from minimum to maximum in the ratio of 1:4 or 1:5, the regulation of the stream over more than one year is of especial benefit to irrigation in years of deficient discharge.

This solution is not so simple in cases such as in the Green River Valley or on the Deschutes River, where the irrigated lands lie above the power development. Here, a compromise such as just suggested is necessary. It is aided by the fact that proper irrigation would have some reservoirs above the irrigated region and, to the extent that water was returned to the stream either by seepage or drainage, the power project would benefit. Such examples are rather unusual, and in the case of the Deschutes the natural regulation of the stream by the lava formation counteracts to a great extent the effects of irrigation diversions on the flow. All power developments on that stream will be by low-head plants. The power demand curve in the Portland region is fairly uniform throughout the year, and as a result of these two factors the power plants will generally be limited in size by the stream flow during the low-water season. The winter excess flow can be stored in the numerous irrigation reservoirs, and this will, in time, act to increase the low-water discharge.

* *Proceedings*, Am. Soc. C. E., December, 1925, Papers and Discussions, p. 1978.

† "Hydro-Electric Developments on the Pacific Coast," *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 803, and *Journal of Electricity*, May 15, 1924.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 814, Fig. 25.

It is true, as mentioned by the author, that in some cases, as on Kings River, California, there is so much irrigation that, in many years, all the flood flow is taken by irrigation ditches with varying priority to the water. Even in such an extreme case, the construction of the Pine Flat Reservoir would make more certain the water supply for all irrigation ditches and would also make possible the development of power on the upper reaches of the river.

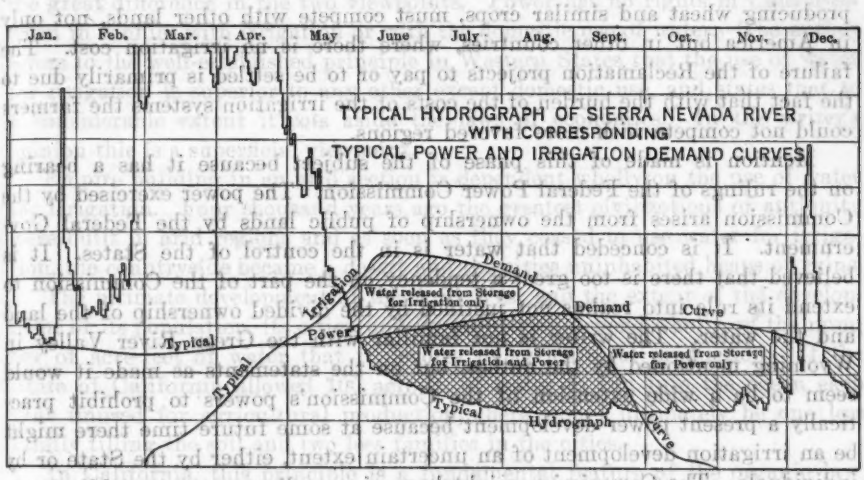

FIG. 17.

The statement by the author that the power demand throughout the year is fairly constant, is only partly correct. On the Pacific Coast there is a wide variation in the type of annual load curve. In rainy regions, such as at Portland, Ore., or at Vancouver, B. C., Canada, the annual peak and mass of maximum demand is in the winter. In California, the reverse is true. In the northern part of the State the power demand is fairly uniform throughout the year, with the peak in the summer. In Central and Southern California the mass of energy is used in summer, the peak load occurs in June or July, and cases exist where the July demand is twice that of February. This is caused by the large irrigation and drainage load and the industries that are of the seasonal agricultural character.

A further correlation between power and irrigation lies in the need of power for drainage pumping. The writer believes that in many, if not most, cases, the drainage of low-lying lands in an irrigation region must be done by pumping, as open ditches are, in most cases, a failure for obvious reasons. They cannot be dug deep enough to produce the hydraulic gradient required to cause the water to run into them.

Many of the projects for irrigation in the West cannot be justified on account of cost and physical conditions. Irrigation has reached its most extensive development in California. On account of the climate, many of the products are of the character that could be termed luxuries—oranges, figs, raisins, and other fruits. These crops bring high prices, but, as a rule, general

irrigation in California that costs more than $75 per acre, capital cost, will not pay. It is difficult to see how irrigation can pay in such regions as on the Upper Deschutes, or in the Green River Valley, where killing frosts occur in eight or ten months of the year. Crops are automatically limited to forage, grains, and hardy fruits. It would not seem that the proposed irrigation on the Columbia River at costs mentioned by the author could ever be justified on a financial basis. It must be remembered that such lands, producing wheat and similar crops, must compete with other lands, not only in America but in other countries, where there is no irrigation cost. The failure of the Reclamation projects to pay or to be settled is primarily due to the fact that with the burden of the costs of the irrigation systems the farmers could not compete with more favored regions.

Mention is made of this phase of the subject because it has a bearing on the rulings of the Federal Power Commission. The power exercised by the Commission arises from the ownership of public lands by the Federal Government. It is conceded that water is in the control of the States. It is believed that there is too great a tendency on the part of the Commission to extend its rule into realms not justified by the divided ownership of the land and the water. The writer is not familiar with the Green River Valley in Wyoming mentioned by the author, but on the statements as made it would seem to be a wide extension of the Commission's powers to prohibit practically a present power development because at some future time there might be an irrigation development of an uncertain extent, either by the State or by individuals. The Commission might have given a license to use public lands and have allowed the State of Wyoming to adjust the use of water as between power and irrigation, that being the function of the State which owns or controls the water, the vital element in both cases.

The problems arising from the dual use of water are complex and grow more so. If treated without prejudice as to either, it can generally be found that power and irrigation are mutually interdependent and can in most cases be made to work together. Colonel Kelly is to be congratulated in having presented a paper in which the most important problems of a wide region are set forth in a clear and comprehensive manner.

FRED. H. TIBBETTS,* M. AM. SOC. C. E. (by letter).†—This paper is of great and timely interest in developing the viewpoint of the engineers of the Federal Power Commission regarding the principles of co-ordination of irrigation and power. However, the author's statements regarding the superiority of the use of water for irrigation and particularly his statement that this well-established principle in the Western States "sets aside the laws of economics" is but a reluctant and half-hearted admission of the limitations under which the engineers of the Federal Power Commission must of necessity work when irrigation and power interests are in conflict.

Of the total estimated installed horse-power covered by licenses of the Federal Power Commission outstanding as of June 30, 1924, 47% was included in projects lying within the State of California. The somewhat different view-

* Civ. Engr., San Francisco, Calif.
† Received by the Secretary, July 8, 1925.

point of the State of California has already been reported to the Society* by Everett N. Bryan, Assoc. M. Am. Soc. C. E., whose position as Office Engineer of the Division of Water Rights of the Department of Public Works of the State of California, closely corresponds to that formerly held by Colonel Kelly with the Federal Power Commission. The very title of Mr. Bryan's paper, "Hydro-Electric Power as a By-Product of Agricultural Storage", epitomizes the great difference in the two viewpoints. Power has no rights in California when in conflict with irrigation or with the domestic use of water. The author refers to the well-established principle in Western States that the use of water for irrigation is superior to any other except domestic use, and states that to a considerable extent it sets aside the laws of economics. In the writer's opinion this is a superficial viewpoint.

Empire building in an arid section is dependent wholly on the use of water for irrigation. Four thousand years ago the greatest civilizations of antiquity were built in arid regions and as soon as they ceased to use water for irrigation, the countryside became a desert and the cities uninhabited heaps of ruins.

The ultimate development of the Arid West and the extent of the development of its population, its cities, and its industries, depend wholly on the number of acre-feet of water that it devotes to agricultural production. If the State of California allowed 100 acre-ft. of water to run into the ocean each year unused for agricultural production, there would ultimately be one less family tilling the soil and two less families in the cities.

In California, this principle is a fundamental feature of the organic law, having been embodied in a forceful declaration of policy by enactment of the Legislature, confirmed by popular vote. The Water Commission Act approved by a referendum vote of the people on November 3, 1914, and as amended since then, contains the following statement:

"It is hereby declared to be the established policy of this State that the use of water for domestic purposes is the highest use of water and that the next highest use is for irrigation. In acting upon applications to appropriate water the commission shall be guided by the above declaration of policy."

The policy so emphatically stated has been continuously observed by the State authority having jurisdiction over the distribution of the State's water resources and has been supported on all occasions for ten years by the State Department of Engineering and Irrigation.

The trend of development of the State's water resources in accordance with this policy is indicated in Mr. Bryan's paper, in which he described several new agricultural developments in California where power is to be produced in connection with storage works created primarily for irrigation supply. Reservoir releases are to be regulated entirely in accordance with irrigation requirements. Such power as is produced, although of great importance and value, will be merely incidental to the main purpose.

It is the writer's belief that the intent as well as the letter of the Act by which Congress organized the Federal Power Commission, makes it mandatory upon the Commission to incorporate in its permits the same policy. Section 9 (*b*) of the Water Power Act reads as follows:

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 910.

"That each applicant for a license hereunder shall submit to the Commission * * * satisfactory evidence that the applicant has complied with the requirements of the laws of the State or States within which the proposed project is to be located with respect to bed and banks and to the appropriation, diversion, and use of water for power purposes and with respect to the right to engage in the business of developing, transmitting, and distributing power, and in any other business necessary to effect the purposes of a license under this Act."

Practically all the important new irrigation developments in California are by irrigation districts, which are municipally organized as State agencies and hence it is also important to note that Section 7 of the Federal Power Commission Act states:

"That in issuing preliminary permits hereunder or licenses where no preliminary permit has been issued and in issuing licenses to new licensees under Section 15 hereof the Commission shall give preference to applications therefor by States and municipalities * * *."

Finally, Section 27 of the Federal Power Commission Act states:

"That nothing herein contained shall be construed as affecting or intending to affect or in any way to interfere with the laws of the respective States relating to the control, appropriation, use or distribution of water used in irrigation or for municipal or other uses, or any vested right acquired therein."

Under these provisions of the Federal Power Act and of the Water Commission Act of the State of California, the writer can see no possible interpretation of the laws other than a mandatory direction to the Federal Power Commission to adopt the State's policy of irrigation superiority and, when irrigation and power applications are in conflict, especially irrigation applications by the State or political subdivision thereof, to cease attempts to determine relative values of kilowatt-hours and acres of irrigated or arid land. The apparent intention of the Federal Power Commission to act only in cooperation with the State authorities has been further assured by the Commission* as follows:

"The development of water power on the lands of the United States requires the approval of both the State and Federal Governments, the former granting the right to use the water, the latter the right to use the land. Under the Federal Water Power Act a license may not be issued until an applicant has secured its water rights from the appropriate State authority. Under such circumstances it is apparent that development can proceed only when the State and the Federal Governments take corresponding action."

The second and principal part of the author's paper is a series of examples of action taken by the Commission on various streams, three of which are California streams. It is of chief interest to the writer because of the conspicuous absence of the Yuba River on which is the most important example of conflicting irrigation and power interests and with regard to which the Executive Secretary of the Federal Power Commission on June 6, 1925, stated his purpose to recommend the rejection of the irrigation applications in favor of the power applications. This recommendation, in the writer's opinion, is in complete violation of all the principles stated in the first part of the paper

* Fourth Annual Rept., Federal Power Comm., 1924, p. 10.

and this, quoting the words* of Arthur P. Davis, Past-President, Am. Soc. C. E., regarding the Colorado River, "in the face of the mandatory provision of the law that the Commission shall give preference to applications from municipalities" and in "defiance of law" (Federal Power Commission Act).

In the three examples of California streams discussed by the author the Trinity and American Rivers were thoroughly investigated by a Special Board which included a representative of the State Department of Engineering and Irrigation and which, therefore, embodied in its findings the State's irrigation policy. In the third case, the Kings River, action was postponed until such time as the applicant had complied with the requirements of the laws of the State of California with respect to appropriation, diversion, and storage of water. In all three cases, the action of the Commission was in thorough accord with the adopted policy of the State.

The strategic position of the Yuba River makes it one of the most important streams in California. It is in a region of the heaviest precipitation in the Sierra Nevada Mountains (in the central part of the State), is near the main centers of urban and manufacturing development, and lies opposite large bodies of the best, and as yet undeveloped, agricultural land in the State. It has excellent reservoir sites so high up in the mountains as to be particularly valuable for power development. It was the scene of the spectacular fight fifty years ago by which the agricultural interests of California in the Supreme Court of the United States checked hydraulic mining, the then leading industry in the State; and for the past four years it has been the battle ground of an exceedingly bitter fight between power and agricultural interests. The power and mining interests were represented by the Excelsior Water and Mining Company and by the Yuba River Power Company, the latter in particular a strongly organized and well conceived project for developing in a comprehensive way a water-shed of about 1 200 sq. miles, with applications before the Federal Power Commission for the development of 227 000 h.p. The agricultural interests were represented by the Nevada Irrigation District municipally organized under the laws of the State of California and proposing to irrigate about 150 000 acres of foothill lands lying, in the main, between elevations of 1 000 and 3 000 ft.

The Irrigation District by co-operation with the Pacific Gas and Electric Company, the largest going concern in Central California, proposes to develop as much of the power resources of the river as it is able and still deliver the water at the relatively high elevation at which its lands are situated. Obviously, it will get less kilowatt-hours per acre-foot of water stored at the head of the Yuba River watershed than the Power Company, whose point of delivery is below the District's lands. This latter project is referred to in Mr. Bryan's paper,† where the irrigation features, if any, of the Yuba River Power Company are referred to as a by-product of power, as follows:

"They [referring to by-product power plants] are even distinctly different from such composite developments as the project of the Yuba River Power Company on the Yuba River, where irrigation, mining, and power are to

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 393.
† *Loc. cit.*, p. 911.

benefit jointly, and in which case if the benefit to either enterprise may be considered a by-product, it is the benefit from stored water delivered for irrigation."

The conflict centers particularly around the Jackson Meadows Reservoir site on the Middle Yuba River at an elevation of 5 800 ft., tributary to which there is an average run-off of 146 300 acre-ft. To the Yuba River Power Company this storage site is of even greater advantage than its water yield would indicate because of its high elevation and its efficiency in regulating the run-off from the lower part of the water-shed. The Irrigation District claims that without this water-shed, from which it derives more than one-half its mountain water supply, it can never obtain sufficient water to irrigate its lands.

The Irrigation District has already contracted with the Pacific Gas and Electric Company for the sale of its power potentialities in a contract (approved by the State Railroad Commission of California and by the Irrigation District Bond Certification Commission) by means of which it will derive a revenue of about $400 000 per year. This contract and this revenue is dependent chiefly on the Jackson Meadows waters in conflict and the District is economically unfeasible and cannot even begin development without these waters. The Power Company on the other hand, while demonstrating that the waters in conflict would be of disproportionately great value to its project, has repeatedly admitted that they were not absolutely essential.

All these facts are matters of record during four years of hearings, largely joint hearings between the State Division of Water Rights and the Federal Power Commission. As the result of these hearings and after the most thorough investigation, the State Division of Water Rights on May 19, 1925, granted the waters in contest to the Irrigation District, confidently assuming that the Federal Power Commission, as by law required, would immediately follow suit.

The District has voted bonds to the extent of $7 250 000 for the construction of its works; its financial soundness is insured by its contract for the disposal of its power resources; its economic feasibility and necessity have been testified by formal resolution of the State Bond Certification Commission; and it was actually ready to proceed when it received on June 6, 1925, the most astonishing news that the Executive Secretary of the Federal Power Commission proposed to recommend the rejection of its application for storage reservoir sites or diversion from the Middle Yuba of waters which had already been granted it by the State of California.

The writer believes that the principles involved in this conflict of Federal and State authority are of vastly more far-reaching importance than either of the two other projects to members of the Society, to the State of California, and to the Nation. Governmental developments within the last twenty years have taken the form of steadily multiplying officials and commissions with overlapping authority. A National Cabinet officer, Herbert Hoover, Hon. M. Am. Soc. C. E., has probably foreseen more clearly than any one else the fateful results of this tendency if it is allowed to proceed unchecked. He is also one of the few men in politics big enough to recommend decreasing the author-

ity of his own Department. In recent speeches on the Pacific Coast he has come out as strongly opposed to further Federal regulatory authority and has stated that the greatest need at Washington is to decentralize the National Government and to pass back into the hands of the people the responsibilities which have been gradually taken from them. The impassé which has been so long foreseen and dreaded by students of this tendency may have at last arrived in California. The Nevada Irrigation District, a political subdivision and an executive agency of the sovereign State of California, the rights of which to its water resources have never been questioned, has been awarded a portion of those public resources by State authority and the Federal Power Commission is denying that State agency the use of Government lands. Unless this conflict can be quickly resolved, the Federal Power Commission will have lost its usefulness and the development of important resources, at least in California, will be threatened with the same stagnation that existed prior to the formation of the Power Commission.

BARRY DIBBLE,* M. AM. SOC. C. E.—No argument is required to establish that agriculture is the basic industry of civilization. Before the beginnings of history, the density of population in some areas had become so great that men could not depend on wild supplies of food. The earliest civilizations that have left any record were founded on cultivation of food crops. In Mesopotamia, the staples were wheat, oats, and barley; in China, rice; in Mexico, Indian corn and maize; in Peru, the potato. It is noteworthy that much of this primitive agriculture was under irrigation.

It is in no sense a disparagement of present-day accomplishments in engineering and industry to class most modern devices as luxuries. If all electrical devices were destroyed to-morrow, mankind would continue to live and function and soon adapt itself to the changed conditions as it did before it had electricity at its service. Strange as it may be, people are willing to and do pay greater rewards to the producer of luxuries than to the producer of necessities. The more nearly a product or a service becomes a necessity, the more critical they are of its price and the profits.

So it is that the farmer thinks he has been overlooked in the distribution of prosperity; so it is that should food prices rise, tariffs will be removed or the farmer's profits in some way brought down; so it is that hydro-electric power development has proved more profitable than irrigation farming.

Nor should it be overlooked that food production in the back country has much to do with the growth of manufacturing industries and centers of population. The electric business feeds on the centers and the centers feed on agriculture. Perhaps it is a law of compensation that justifies Western agriculture in collecting some toll when irrigation water is used for power production.

Many irrigation projects in their development and growth present favorable opportunities for power production—often as a by-product. Usually, the water users are alive to the possibilities and if a market is available the question presents itself whether the power house should be constructed by the project

* Cons. Engr., Redlands, Calif.

or whether the power rights should be sold to a power company. In some cases, most or all of the power can be efficiently used for irrigation pumping on the project land. Generally, however, the power must be sold and the price at which it can be sold governs the feasibility of the power development. The primary objective of the water users is then to obtain the greatest net revenue without interfering with the application of water to irrigation.

If the irrigation district sells its power rights, its investment is reduced and the financial risk correspondingly lessened; the district is kept out of the power business, with its more or less technical requirements. On the other hand, the purchaser of power rights expects to make a profit and will discount the risks and uncertainties of the future as far as he can see them. Such an arrangement must be based on the contract in which the rights of both parties are set forth. The district will try to protect the water rights of the irrigationists; the power company will try to obtain a free hand to use water when most needed, for power purposes.

Cycles of wet and dry years, changing conditions in irrigation and power demands, may give important and unanticipated economic advantage to one or the other of the parties. The power production may either be far greater than expected or the power company may find itself paying for something it does not get.

Bitterness and jealousies between the contracting organizations are likely to be aroused, and the contract will probably find its way into the Courts. If the power house is built and operated by the water users and the energy sold on a production basis, they immediately become interested in securing the largest and most saleable output from the water available. When the pinch comes, in years of drought, they will be far more ready to co-operate to get the best results than if they receive no benefit. It is much easier to draft a contract that will prove satisfactory, when the interests of the parties run along parallel lines.

The modern hydro-electric station, properly designed and installed, is not difficult to operate efficiently and satisfactorily. Often, the irrigation district can handle the plant with an organization not much different from that which would be required to care for the irrigation work alone. The hydro-electric construction involves the irrigation district in larger investment and in certain risks of the business. However, the district is in better position than any one else, to use the investment to the best advantage. The risks must be taken by some one. If the right to develop power is sold, they come back to the district by lowering the consideration paid for those rights.

The Minidoka Project, in Idaho, and the Salt River Project, in Arizona, are examples of the better co-ordination that can be secured between power and irrigation, when the power is owned and generated by the irrigation interests. In each instance, the use of structures and regulation of flow have been materially modified to meet the needs for power, and to increase the revenue. Irrigation has not suffered and yet it is certain that the water users would bitterly have fought the proposal that some outside company be allowed to do the things they have done.

Generally, the best results from both economic and other standpoints will be secured where the ultimate water users themselves are directly interested in seeing the water put to the fullest beneficial use.

MARKHAM CHEEVER,* ESQ.—It is eminently fitting that a discussion of this paper should take place in Utah where pioneer work has been done both in irrigation and in the generation and distribution of hydro-electric energy, and where both uses of water have been co-ordinated to such a great extent as to provide a high degree of beneficial utilization of the water resources.

The early settlers in Utah were the first in the United States to practice irrigation on an extensive scale and the general condition of prosperity throughout the irrigated areas is evidence of the economic soundness with which the projects were conceived. Likewise, much valuable pioneer work in the development of hydro-electric energy from water power was done in Utah. Approximately thirty years ago construction was commenced on hydro-electric plants in Provo, Ogden, and Big Cottonwood Canyons, and, in 1897, the first commercial transmission line in the country to operate at so high a voltage as 40 000 was placed in service to distribute energy from the Provo Canyon plant. Since that time, the development of water power has been on a substantial scale and an adequate supply of hydro-electric energy has been made available over wide areas for lighting cities and towns, operating irrigation pumping plants, and developing industries. It is especially interesting to note that electrical service has been made available to a larger proportion of the farms in Utah than in any other State. According to the census of 1920, which represents the latest authoritative source of information, Utah was at the head of the list with more than 34% of the farms receiving electrical service as against 7% representing the average for the whole United States.

There is in the vicinity of Salt Lake City a striking example of successful co-ordination between irrigation and power development. Bear River is the largest stream flowing into Great Salt Lake. It rises on the northerly slope of the Uintah Mountains, in Northeastern Utah, flows northward through Wyoming, enters Southeastern Idaho, and after making a turn around the northerly end of the Bear River Range flows southward into Utah and discharges into Great Salt Lake. In Southeastern Idaho it flows near, but not through, Bear Lake, a body of water having an area of 110 sq. miles. By means of canals and other extensive diversion and control works, including a pumping plant, the lake has been converted into a reservoir of capacity adequate to equalize completely the flow of the river below that point, not only during any one year, but also throughout a long series of years.

The total fall between Bear Lake and Great Salt Lake in a distance by river of approximately 200 miles is 1 720 ft. The river flows alternately through open valleys and canyon sections. The valleys are irrigated and in the canyons it has been possible to utilize for power development a substantial part of the total fall in the stream. The use of Bear Lake Reservoir has enabled the production of a large amount of firm power, and at the same time all irrigation requirements in the river basin have been met. During

* Gen. Supt. and Chf. Engr., Utah Power & Light Co., Salt Lake City, Utah.

the exceptionally dry year of 1919 full crops were raised in the Bear River Basin, and it was estimated by a representative of the United States Department of Agriculture that in that year the value of crops saved by the release of stored water from Bear Lake amounted to between $4 000 000 and $5 000 000.

Although the vital importance of irrigation to the arid States is unquestioned, the value of an adequate and reliable supply of electrical energy must not be overlooked. In addition to the obvious advantageous use of electricity for pumping irrigation water, the widespread availability of electrical service from large interconnected systems has stimulated industries which depend on agriculture and which, in turn, create a market for farm products. Electrical energy has revolutionized industrial processes, has assisted greatly in conserving labor and has produced domestic conveniences which have now become necessities. New and enlarged uses for energy are constantly arising, and it is inevitable that the electric power industry will become of increasingly great importance in the economic development of the United States.

Difficulties in co-ordinating irrigation and power projects are in many cases imaginary rather than real. Misunderstandings arise because of lack of reliable and adequate data, or because of failure to analyze the information correctly. If, in each case, the full facts are made available and are viewed disinterestedly from the standpoint of economic practicability of the projects involved and of best results in the general development of the territory served, the solution of the problem of co-ordination will not be difficult. It should be remembered, however, that no two situations are alike in either physical or economic characteristics and no general formula can be previsioned and thereafter applied to all future problems of co-ordination if the greatest beneficial results are to be obtained from developing the water resources for both important uses.

M. C. HINDERLIDER,* M. AM. SOC. C. E.—The subject of co-operation in irrigation and power development is not of prime importance to Colorado at this time and this is largely true of the States in the Rocky Mountain region, due primarily to three factors:

First.—The wide monthly and yearly fluctuations in run-off in the streams of the Rocky Mountain region would require the use of extensive storage systems for the proper regulation of flow to meet the demands of power development or to justify expenditures for such purposes on an economical scale.

Second.—Although an extensive system of storage reservoirs with an aggregate capacity of 2 400 000 acre-ft. is now in use in Colorado, such reservoirs are used almost wholly for irrigation and cannot be converted to power use without secondary storage which, under present conditions, would not be economically feasible.

Owing to the necessities of irrigation which is of paramount importance in the semi-arid States, stream run-off over periods of many months must be stored and held in reserve against the demands occurring later in the summer, which demands usually cover only two or three months.

* State Engr. of Colorado, Denver, Colo.

Additional difficulties arise over the system of laws and administration wherein priority of claim for irrigation use must be observed regardless of where such claims occur on a stream. The time is anticipated, however, when through proper legislation present priority requirements for irrigation uses will be so modified as to remove some of the obstacles now faced by the promoter of hydro-developments.

Third.—Such developments are not more extensive in the Rocky Mountain region: (1) because of the great deposits of cheap coal which are found throughout this part of the United States; and (2) because of the attendant cost of transmission over sparsely settled areas. In Colorado, for the present at least, investigations have shown that power may be supplied more economically by means of steam-generating plants than from available water power sources.

These conditions will doubtless be modified with the passing of time, when due to greatly increased density of population and to attendant demands, together with reduced cost of transmission, hydro-power possibilities, aggregating several million horse-power, will be utilized through proper legislative provisions, permitting of a high state of co-ordination and co-operation between the dominant irrigation interests and power possibilities. Such co-ordination must of necessity come about rather slowly in order that existing corporate and individual rights may not be unduly disturbed in the process of giving way to the greater demands of the public.

THE HEXAGONAL SLAB DESIGN OF CONCRETE PAVEMENT

Discussion*

By E. G. Hooper, Assoc. M. Am. Soc. C. E.

E. G. Hooper,† Assoc. M. Am. Soc. C. E. (by letter).‡—The problem of pavement design has pre-empted the thought of many practicing and research engineers ever since the automobile became such an important economic factor. Many novel ideas have been developed and presented as the solution of that problem, but as yet none has proved more than partly satisfactory in meeting demands under widely different and varying conditions. A full appreciation of the limitations of the several more generally adopted plans, together with a thorough knowledge of the loads, structural material, and soil conditions in a given locality, makes it feasible to select a reasonably satisfactory type. The outstanding factor militating against a satisfactory general design is a widely varying undependable supporting sub-grade. Until some means is found to reduce materially the number of different critical conditions that can develop under a pavement no satisfactory structural study can be made, as that requires an established maximum loading, a knowledge of the behavior of structural material and type of structure, and a certainty of permanency in the nature and positions of the supporting reactions. The only thing the writer can imagine that will obviate this fundamental difficulty is to provide supports that will carry the reactions to a soil level below that subject to disturbance. A design has been evolved along this line that is practical, the cost of which is not materially above that of present pavements. This discussion, however, will be confined to points of agreement and disagreement in Mr. Perry's paper.

The writer's comment, elaborated subsequently, may be summed up briefly, as follows: It is agreed that (1) corners of pavement slabs are the seats of greatest weakness; (2) corners of hexagonal slabs are stronger than those of rectangular slabs; and (3) improperly planned doweling across joints may produce serious spalling. On the other hand, it is objected that (1) increase in the number of corners by use of hexagonal slabs may offset to a degree the advantage of individual corner strength; (2) the position of corners with reference to traffic lines in the hexagonal slab arrangement proposed subjects these weak places to unusual punishment; (3) the regular zigzag line or joint

* This discussion (of the paper by Lewis A. Perry, Assoc. M. Am. Soc. C. E., published in November, 1925, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Asst. Prof., Civ. Eng., New York Univ., New York, N. Y.

‡ Received by the Secretary, November 5, 1925.

making an angle of 30° with the center line of the road is a possible source of confusion to drivers; (4) temperature curling at corners that are coming continuously and suddenly under load is likely to aggravate conditions beyond normal; (5) absence of shear connections with adjoining units permits at times relative movement, or displacement of a permanent nature, at joints so that impact, otherwise negligible, may become a formidable factor; and (6) on soil which very little water can reach and from which that little is quickly drawn, as in built-up and sewered communities, the weaknesses of any of the slab designs would not develop to any great degree.

As Mr. Perry states in his excellent presentation the common rectangular slab construction develops its greatest weakness at the corners. The importance of strengthening these corners is definitely recognized, more particularly since the publication of results of, and conclusions from, the "Bates" and the "Pittsburg" road tests. Various means have been adopted for making, directly or indirectly, such points as strong as the remainder of the unit. Mr. Perry approves of thickening the edges as he has incorporated that idea where he, of necessity, has right angles; but he objects to the use of dowels or tongue-and-groove joints for transferring some of the load to adjoining members.

Objection to dowels may be partly justified. An insufficient number and size of bar together with an open joint will inevitably produce spalling which may eventually nullify the expected benefits from the steel. However, as such spalling is due to excessive bending of bars across an open joint, or is caused by closure of the joint by temperature, where a dowel is bent or out of alignment it may be corrected by use of bars of proper size and number, to make the bending negligible and the face-bearing area adequate. The writer believes expansion from corrosion is negligible because air and moisture must have access to the embedded metal; even the sliding ends of bars are usually wound with an adequate protective covering.

Tongue-and-groove joints are effective in transferring shear or load from one unit to another and the percentage thus transmitted is a function of the relative deformation of the adjoining units. Exact calculations cannot be made because of the ever-present bugaboo—variable and indeterminate soil resistance—which renders it impossible to predict slab deformation. Numerous service tests have given favorable impressions of carefully designed and made joints of this kind, and until other acceptable tests prove otherwise they may logically be accepted as satisfactory.

It is readily acknowledged that full hexagonal slabs are stronger at the corners than rectangular ones, nevertheless those corners remain a weakness. The use of half hexagons for pavements, as recommended by Mr. Perry, of necessity, retain two of the taboo right angles at critical points. In place of two other right angles along the center line, he substitutes three 120° angles, all three of which are situated so that the left-hand wheels of every vehicle traveling the right-hand side of the road will traverse the ones on that side, and in such a manner as to apply a force comparable with a critical load in

case of heavy trucks. Vehicles traveling down the center will likewise traverse the corners, so that the weakest points of the pavement are suffering punishment all the time, a condition not duplicated in the usual rectangular construction. It is conceivable, therefore, that, in spite of the apparently greater strength, the hexagonal slab would break down under as light or lighter traffic than the rectangular slab.

The ordinary "geology, topography, climate" combination for any particular locality produces such a great variation in the supporting value for the same soil that a number of critical design conditions develop. Unless all these critical conditions have been reproduced or approximated for the given case no set of tests can be said to prove the fitness of a design, even for the one case, much less for general use. Mr. Perry's tests show the comparative strength of hexagons and rectangles with the conditions of load and support shown and described in his paper. As pointed out before, in practice few loads will pass over rectangle corners as compared with those of hexagons in the plan proposed. The test results, therefore, may not be taken as fairly indicative of relative service values.

Absence of ties between the pavement units makes a collection of free slabs, which, at the worst, where soil conditions are very unsatisfactory, is comparable with a field of floating ice cakes. As already stated the best sub-grades serve equally well for any of the better slab types; it is the poorer sub-grades that require the special treatment so many engineers are trying to develop. The action of a soft soil under pressure is to an extent comparable with water. To develop the illustration, if we step on one corner of a floating ice cake that point will go down and the opposite side will rise; in other words, it will tip. The amount of tip, the amount of depression of the corner, is greater for the smaller cakes. If we step from one cake to the next, the one tends to come back as that next cake tips. There is a difference in level between adjoining units during transit of loads, a condition that would produce impact with a rolling load. In the case of pavement units similar vertical movement is bound to occur, its magnitude depending on the stability of the particular sub-grade, the size of the slab, and the number of load repetitions. Although such movement may be insignificant, repetition may develop permanent difference in level. Moreover, the impact developed as a horizontal component against an elevated edge will most probably work the slabs apart laterally even if reasonably good shoulders are maintained. For these reasons the writer considers independent action of slabs highly undesirable.

Another point of minor importance is the comparative amount of joint filler for the two types of pavement. For a given area the perimeter or joint length of a hexagon is the less, and the same may be said for the half hexagon. Were the usual transverse joint spacings the same as are required in the "hex" design, it would be correct to claim a saving in joint material. As a matter of fact, however, there are enough fewer such joints in the usual rectangular design to eliminate any such saving. This is another instance where comparisons under different conditions modify or change conclusions.

An advantage of the small units, offsetting to a degree the disadvantage, is greater freedom from cracks developed in the early stages of setting when volume changes due to moisture variation and temperature occur. As such small cracks are fruitful sources or beginnings from which develop the large disruptive breaks, it is obviously very desirable to reduce them to a minimum.

In discussing at such length the practicability of hexagonal slab design for highway pavements, the writer has had in mind that he should make his viewpoint clear. Although many of the premises, or conclusions, or both, may not be approved he believes that a consideration of them all will be helpful in the final analysis.

STREAM POLLUTION

A SYMPOSIUM

Discussion*

By Harrison P. Eddy, M. Am. Soc. C. E.

Harrison P. Eddy,† M. Am. Soc. C. E.—The members of the Sanitary Division are indebted to Dr. Frost and the other investigators of the United States Public Health Service for the fundamental scientific work which they have been doing over a period of many years. Doubtless, they enjoy some satisfaction and encouragement in knowing that their investigations have provided much of the supporting data and evidence used by the Engineering Board of Review in its study and in the preparation of its report on the disposal of the sewage of the Sanitary District of Chicago.

Thus far, the sewage has been disposed by dilution, only an insignificant part having received treatment prior to the discharge into the Canal System. Among the interesting conditions is the unique fact that in the summer the Des Plaines-Illinois River into which the sewage is discharged is nearly as large at its source as at its mouth. This is due to the great volume of sewage and diluting water discharged from the Main Drainage Canal of the Sanitary District into what may be termed the head-waters of the Des Plaines-Illinois River the natural flow of which is relatively small at that season of the year. At the point of confluence this flow is usually a few hundred cubic feet or less per second, whereas the discharge from the Main Drainage Canal has been as large as 10 000 cu. ft., of which about 10% was sewage.

Another matter of great interest developed during the investigations of the Board of Review is the effect of the temperature. Dr. Theriault's paper‡ shows how the temperature has a marked effect on the volume of diluting water required to maintain certain conditions in the river. The discharge of polluting matter or sewage into a river during the cooler season of the year may be compared with refrigeration. Meat is placed on ice to prevent its decomposition and change. If sewage is discharged into a river during the winter the rate of change will be extremely slow, with the result that the volume of diluting water required to maintain a certain percentage of saturation with oxygen will be relatively small. This condition has a very important bearing in many disposal problems and advantage may be taken of it to reduce the cost of treatment of industrial wastes and sewage in many cases in the

* Discussion continued from December, 1925, *Proceedings*.

† Cons. Engr. (Metcalf & Eddy), Boston, Mass.

‡ *Proceedings*, Am. Soc. C. E., November, 1925, Papers and Discussions, p. 1819.

northern part of the United States. This has been done notably by certain tanneries and paper mills in Massachusetts. It is practicable for such industries to reduce the treatment of their wastes to the removal of settleable solids during the cool season of the year, whereas during the period of high temperature it may be necessary to carry the treatment much further. This principle of more fully utilizing the conditions of Nature in the economical disposal of sewage and industrial wastes undoubtedly will occupy a more important place in the operation of treatment plants in the future than it has in the past.

Another interesting subject, mentioned by Mr. Streeter,* is the effect of sludge deposits in rivers. The study of the Board of Review showed that this is an important factor in the disposal of the Chicago sewage by dilution, as it has been in many stream-pollution problems in other parts of the country.

The effect of temperature upon the oxygen demand exerted by organic matter in the water is not different from the effect upon the oxygen demand exerted by the organic matter in sludge deposits. Solids suspended in the water constitute a traveling load. In winter, those which are carried by the currents from the source to the mouth of a river may not exert a great oxygen demand and they may then finally be disposed of with similar solids which may be deposited on the bed of the river. They also will exert only a relatively small oxygen demand in winter, but as soon as the temperature reaches the point where decomposition proceeds rapidly the rate at which they exert their oxygen demand is greatly increased. Thus, a larger proportion of the oxygen demand of the year's deposit of solids is exerted in the two or three months during which the oxygen demand of the flowing load is also highest. This illustrates the importance in many cases of always removing the suspended matter from sewage and industrial wastes even if the oxygen supply in the river water during certain seasons of the year may be high.

* *Proceedings*, Am. Soc. C. E., November, 1925, Papers and Discussions, p. 1829.

THE EFFECT OF MOISTURE ON CONCRETE

Discussion*

By G. E. P. Smith, M. Am. Soc. C. E.

G. E. P. Smith,† M. Am. Soc. C. E. (by letter).‡—It has been realized for a long time that climatic effects on concrete are of great importance, and that quantitative data adequate for designing and for determining practice in light construction exposed to the weather, such as sidewalks, are lacking. This paper is, therefore, exceedingly welcome at the present time. Much still remains to be done, and it is to be hoped that in laboratories properly equipped for this work, investigations will be continued until the causes of change of volume and the factors which govern variation in the effects of temperature and moisture are well understood. In Southern Arizona and regions of similar climate, where the relative humidity may be less than 20% for many days in succession and where hot, dry forenoons are followed by rainy afternoons, the importance of such knowledge is evidenced by the prevalence of crazing marks and by the large percentage of concrete construction which is disfigured by cracks or which becomes disintegrated until replacement is necessary. Engineers speak of "concrete for permanence", and would be happy to feel assured that the motto is reliably true with respect to even severe weathering conditions.

In 1918, the writer called attention§ to some important failures of concrete pipe lines at Continental, Ariz., and to similar failures in California. At Continental, cement pipe made on a McCracken revolving packer-head machine—pipe of unquestionably high-grade quality—failed soon after being put in service, by longitudinal cracks, in one case by a crack approximately 1 000 ft. long. The pipe had been allowed to stand in the stockyard until it had become thoroughly dry. The differential expansion in the shell due to absorption of water must have produced high tensile stresses, inasmuch as the pipe when tested withstood the stress produced by a pressure head of more than 100 ft. of water. The pipe was too "good", it broke itself; weak, porous, hand-tamped pipe never fails from this cause. In a study of the causes of these failures it was found that a joint of pipe immersed in water gained 5% in weight with an elongation of 0.056 per cent. A joint that had been broken in the internal-pressure testing machine and, therefore, had an open crack on one side, was tested by immersing one-half the pipe, laid flat, in water.

* Discussion on the paper by W. K. Hatt, M. Am. Soc. C. E., continued from August, 1925, *Proceedings*.

† Prof. of Irrig. Eng., Univ. of Arizona, Tucson, Ariz.

‡ Received by the Secretary, October 26, 1925.

§ *Bulletin 86*, Arizona Agricultural Experiment Station, 1918; also, *Engineering News-Record*, Vol. 83, No. 3, p. 113, July 17, 1919.

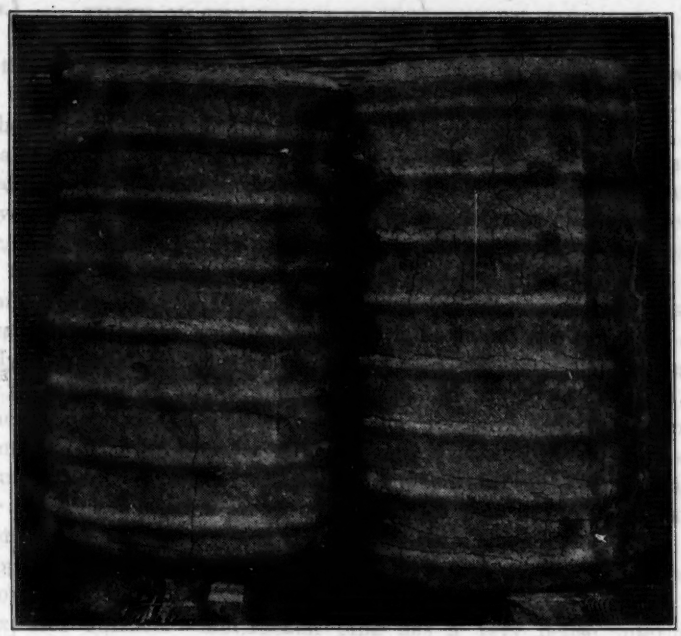

FIG. 25.—CRAZING MARKS AND OPEN CRACKS ON 14-INCH McCRACKEN CEMENT PIPE.

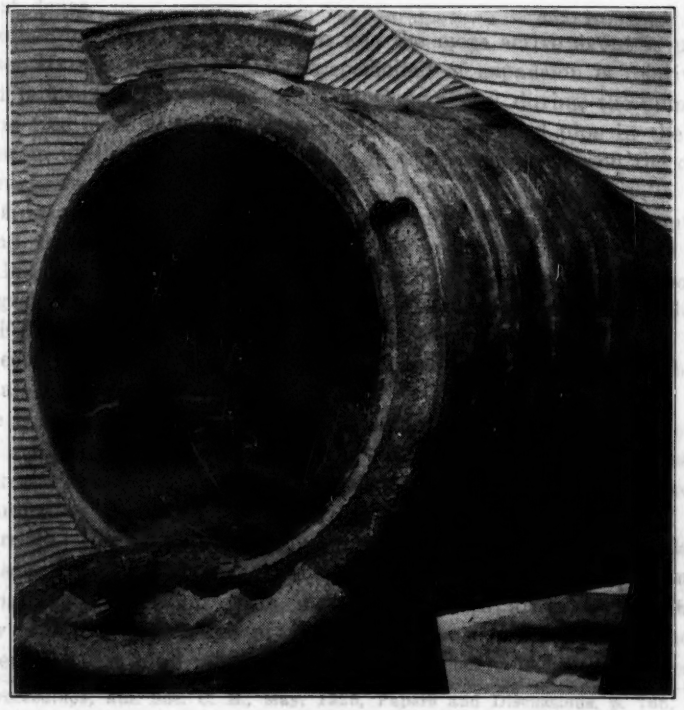

FIG. 26.—VIEW OF THE INTERIOR OF A CEMENT PIPE, SHOWING CRACKS. THE HUBS OR GROOVES BROKE OFF IN SEGMENTS.

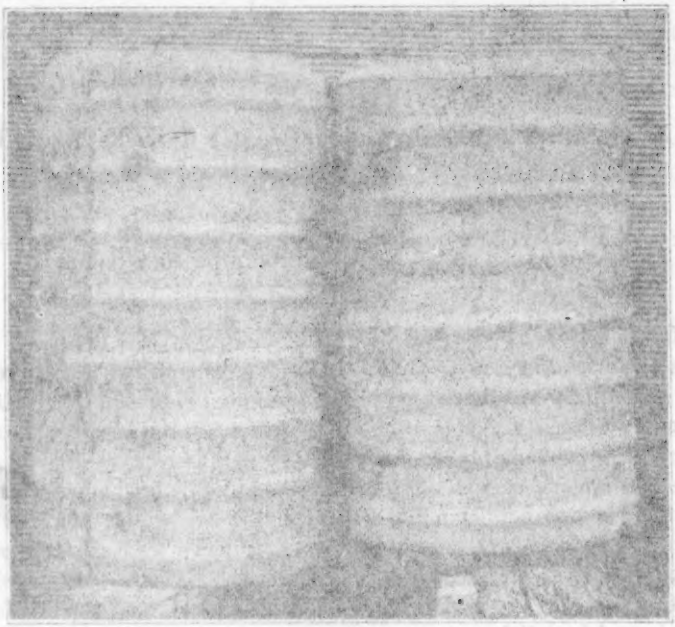

FIG. 25.—CRACKED MARKS AND OPEN CRACKS ON 14-INCH MONACHAN CEMENT PIPE.

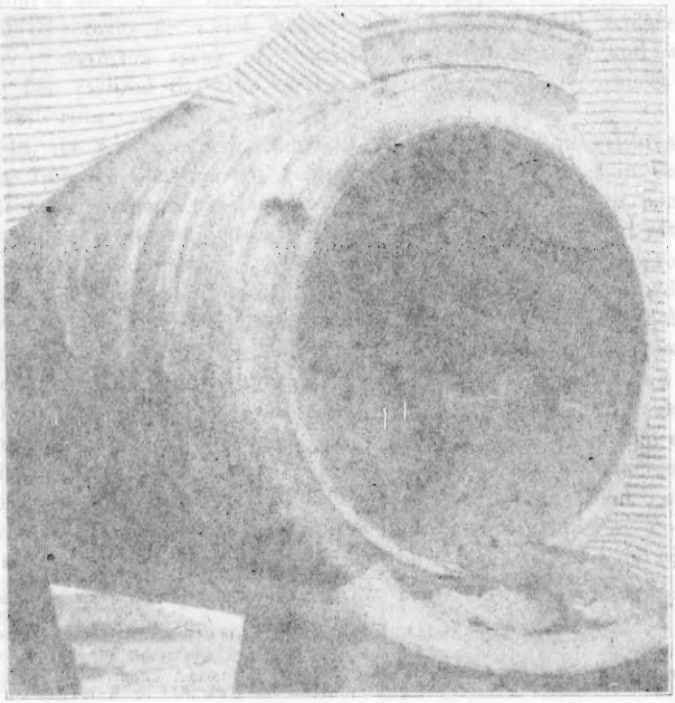

FIG. 26.—VIEW OF THE INTERIOR OF A CEMENT PIPE, SHOWING CRACKS. THE RUNS OR GROOVES BROKE OFF IN SEGMENTS.

With the absorption of water, mostly from the outside, the crack closed rapidly about 0.045 in. and then slowly opened again.

When the pipe-laying program at Continental was terminated, there was a considerable supply of 12-in. and 14-in. and some 16-in. pipes on hand, stacked three tiers high in the stockyard. These pipe have laid undisturbed for seven years exposed to sun and rain. When he examined them recently, the writer, discovered that fully one-half the pipes were in bad condition and that about 10% of the top layer had cracked wide open. Fig. 25 shows the condition of the exterior and Fig. 26 of the interior of one of the pipes. Most of the cracks start at the tongue end. The hubs or grooves break off circumferentially. None of the pipe of this size failed underground, although similar 20-in. pipe failed by longitudinal cracks along the top of the pipe line.

The crazing on the exterior shows that the shrinkage has been both circumferential and lengthwise. On the interior, the circular cracks are more pronounced, and occur somewhat regularly beneath the corrugations or beads of the outer surface, due probably to the fact that the force of expansion is greater in the section beneath the beads than elsewhere.

Apparently the expansion of the outer part of the pipe wall produced the interior cracks and the rapid shrinkage of the outer part caused the crazing of the exterior surface. McCracken pipe has a very dense interior surface. Hence, most of the absorption of water and drying out is through the outer surface. The cracks tend to occur at those parts of the pipe that are relatively the most dense.

Whether temperature or moisture was the more active agent is uncertain. Professor Hatt has shown* that the coefficient of expansion is relatively high at high temperatures, such as occur at Continental during the summer, but with the free circulation of air through the open-ended pipe in the stack, it would seem unlikely that the difference in temperature between the outer and inner surfaces would be great.

Crazing cracks are common, also, on building floors that are continuously dry, and these cracks become progressively more pronounced year by year. Sidewalks, ornamental park benches, and building steps in Tucson, Ariz., suffer greatly from crazing cracks which develop into breaks and disintegration. Fully one-half the concrete sidewalks have a too limited life. Some of these results can be traced to poor workmanship or faulty curing, but by no means all of them.

The U. S. Bureau of Standards has investigated the properties of cement having a high magnesia content.† The effect of varying percentages of magnesia on the expansion when immersed and the contraction when dried indoors was studied by means of 5 by 5 by 45-in. bars. An inspection of the published results indicates that, neglecting two cements that were under-burned and unsound and three others, of the high alumina series, that were manifestly abnormal, the expansion from moist-chamber to air-dry condition varies directly with the percentage of magnesia. The cement used in the pipe at Continental had a high magnesia content, and there is a strong presumption

* *Proceedings*, Am. Soc. C. E., May, 1925, Papers and Discussions, p. 765.

† *Technologic Paper No. 102*, U. S. Bureau of Standards, 1918.

that this had much to do with the failures of the pipe lines and the destruction of the pipe in the stockyard.

The shape of the pipe is such as to bring out to its fullest extent the injurious action of differential expansion. For massive construction and in humid climates, the effects may be inconsequential, but it is probable that for severe conditions the specifications of cement should contain a clause limiting the contraction of 1 : 3 mortar to, say, 0.02% in 1 month, or 0.03% in 3 months, and the methods of conducting tests should be standardized.

It is of great interest to note that Professor Hatt found wide differences in the effect of moisture on expansion and contraction in the six different brands of cement tested. The variation in the coefficient of expansion due to temperature was slight. It would be of value to know the percentages of magnesia in the six brands.

The thermal coefficient stated in the author's conclusions seems to be based on the oven-dry specimen reported in Fig. 6.* The air-dry specimens have much higher coefficients.

In the case of the beams warped by the saturation of the under side, can it not be assumed that a part of the resulting compression on the bottom is relieved by warping, causing compression in the top side, and that tension exists in the beam between the line of saturation and the neutral axis to such an extent that the horizontal initial forces (stresses) are balanced? It may be remarked that, if the conditions shown in Fig. 20(*A*)† are reversed, starting with a saturated beam, the effects on the beam will be much more injurious.

During saturation of specimens of cement pipe at Tucson, it was found that the expansion lags somewhat behind the absorption, suggesting that a change in crystallization is taking place, with the addition of water to the molecule.

* *Proceedings*, Am. Soc. C. E., May, 1925, Papers and Discussions, p. 767.
† *Loc. cit.*, p. 781.

CONTROL OF ODORS FROM SEWAGE TREATMENT PLANTS

Discussion*

By John F. Skinner, M. Am. Soc. C. E.†

John F. Skinner,‡ M. Am. Soc. C. E. (by letter).§—The writer appreciates the value of discussion elicited from engineers in charge of different types of plants and illustrated by conditions in widely separated localities.

Mr. Cohn‖ refers to the sour odor "caused by the fermentation of the great quantities of mash that clog the gas vents." At the Irondequoit plant at Rochester, N. Y., "considerable quantities of grain are found in the grit, the probable source being the wastes from home brewing processes." This material is deposited with the grit and if promptly removed causes no offense. This is one of the reasons why a small grit chamber may be desirable in a plant limited to the treatment of sanitary sewage. Mr. Cohn's observations indicate that care and cleanliness about the plant remove most causes of complaint.

Mr. Keefer's work at Baltimore, Md.,¶ including the confining of gases and their treatment with chlorine, is most interesting and points the way to further experimentation. It is in line with the writer's suggestion.

Mr. Gillespie's odor surveys** are excellent in furnishing comparative results which may be studied in the light of location, exposure, prevailing winds, type of sewage, and care in operation. His statement, "daytime inspections are invariably negative as to odors, except at most within a few hundred feet, whereas odors at evening travel many times as far", corresponds with the writer's observation and his explanation of the phenomena due to convection.

The writer has assumed that any considerable quantities of putrefying industrial wastes should be treated before admission to the public sewers. In general, the expense of such treatment should be borne by the interests involved rather than by the general public.

Mr. Gillespie's observations at El Centro, Calif., indicate how a trained and observant operator may relieve an acute condition by a specific remedy. The plowing under of activated sludge at Lodi, Calif., and its local use as fertilizer, is doubtless the best solution for a small plant suitably situated.

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* Discussion of the paper by John F. Skinner, M. Am. Soc. C. E., continued from August, 1925, *Proceedings*.

† Author's closure.

‡ Cons. Engr.; Deputy City Engr., Rochester, N. Y.

§ Received by the Secretary, November 21, 1925.

‖ *Proceedings*, Am. Soc. C. E., August, 1925, Papers and Discussions, p. 1182.

¶ *Loc. cit.*, p. 1185.

** *Loc. cit.*, p. 1187.

Mr. Hatton's criticism* anent lagooning of activated sludge is interesting. He states that at Houston, Tex., little or no scum has formed, while at Saltley in the Manchester, England, plant he refers to the formation of heavy top scum.

J. C. McVea, M. Am. Soc. C. E., City Engineer of Houston, writes that his sludge lagoons are 18 in. to 10 ft. deep, and of various sizes up to nearly 1 acre in area:

"No appreciable amount of scum forms on the surface * * * and it will be necessary for them [lagoons] to stand several months before they are entirely dry. Odors are not noticeable during the normal operation of the lagoons, but upon allowing them to stand for several months and to dry out a very offensive odor is created upon again filling with sludge unless the old material is entirely removed."

This is with a range of temperature from 50 to 105° Fahr.

Mr. Gillespie writes that at Pasadena, Calif., the activated sludge was lagooned 2 or 3 ft. deep in a sandy river bottom:

"The sludge soured badly and made a terrific nuisance, reaching about 6 000 ft. away where it made a nuisance to some 7 000 people. Examination of the beds seemed to indicate that they remained sour for at least a month * * *. No scum formed on the surface of the lagoon. It was rather a pasty sludge. There was a thin, dry crust which was evidently not sufficient to confine the odors."

The lagooning has now been discontinued and the present practice is to coagulate the sludge with alum, dry on an Oliver filter, and bury in deep trenches. The latitude of Pasadena is 34° N.

At Houston, the lagoons are operated continuously at considerable depth practically as open septic tanks. In Pasadena the odor from sour sludge in shallow lagoons became unbearable. Odors from lagooning apparently depend on the condition of the sludge, the temperature, the depth of supernatant water, the formation of scum, and the method of operation. The writer, therefore, feels justified in his statement† that, "In general, lagooning should only be adopted in winter, or in remote locations where no nuisance will be occasioned to neighbors."

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* *Proceedings*, Am. Soc. C. E., August, 1925, Papers and Discussions, p. 1191.

† *Loc. cit.*, May, 1925, Papers and Discussions, p. 850.

HIGH SPECIFIC SPEED HYDRAULIC TURBINES IN THEIR BEARING ON THE PROPORTIONING OF THE NUMBER OF UNITS IN LOW-HEAD HYDRO-ELECTRIC PLANTS

A SYMPOSIUM

Discussion*

By Messrs. W. M. White and Lewis F. Moody†

W. M. White,‡ M. Am. Soc. C. E.—The papers contained in this Symposium§ on the important subject of high specific speed hydraulic turbines, have been very comprehensive. The speaker had the pleasure of visiting the Lilla-Edet plant to which reference has been made‖ and was greatly impressed with the tremendous size of the conduits leading the water to and from the wheels. The construction has been considerably delayed due to labor troubles. The Verkstaden Works in Kristinehamn, Sweden, the builders of the turbines for this plant, extended an invitation to the speaker to visit the works and he had an opportunity of seeing the turbines in course of construction. There is more need for the movable vane runner in Europe than in America, because here such a network of power lines exists that nearly every plant constructed can be operated to its full capacity as long as water is available.

Mr. Moody¶ has given a formula which throws additional light on an interesting and important subject.

Mr. Jessop's paper** has conformed more exactly to Mr. Orrok's suggestions as to scope than any of the others and the speaker is sure that his presentation will be of value.

Mr. Nagler†† has done valuable work and has made a real contribution to the art. His type of runner enables the development of lower heads than would otherwise be possible.

The high-speed runner, as designed by Mr. Moody, seems to be a type which lies in the intermediate zone between the Z-type of Leffel wheel and the four-bladed "Nagler" wheel. It is to be hoped that the Engineering Profession will encourage further development along these lines by giving protection to those

* Discussion continued from October, 1925, *Proceedings*.

† Author's closure.

‡ Mgr. and Chf. Engr., Hydr. Dept., Allis-Chalmers Mfg. Co., Milwaukee, Wis.

§ *Proceedings*, Am. Soc. C. E., August, 1925, Papers and Discussions, pp. 999 and 1050.

‖ *Loc. cit.*, p. 1000.

¶ *Loc. cit.*, p. 1012.

** *Loc. cit.*, p. 1043.

†† *Loc. cit.*, p. 1032.

making inventions and pursuing development work. The only way a manufacturer has of recovering expenditures made for developments such as have been discussed in these papers is by having the exclusive use of the devices thus developed for a reasonable time.

LEWIS F. MOODY,* Esq. (by letter).†—As a supplement to the material contained in the Symposium,‡ on "High Specific Speed Hydraulic Turbines," the writer would call attention to a paper, which appeared shortly after the presentation of the Symposium, by Franz Staufer, of Munich, Germany, entitled "Influences on the Efficiency of Water Turbines."§ A brief abstract of this paper by C. Reindl has also been published.‖

Referring to the writer's discussion¶ of the variation of efficiency with size of turbine, it is interesting to note that Mr. Staufer has taken up this same subject along similar lines, has developed a formula of the same form, and has arrived at the same numerical value of the exponent of the ratio of diameters appearing in the formula.

Mr. Staufer mentions the point noted by the writer that the change in efficiency of water turbines as affected by diameter and head, as first formulated by Camerer, was based on the formula of Biel. Instead of using the Biel formula, Mr. Staufer, proceeding along the same lines as the writer, has based his work upon the exponential expressions for the loss of head in fluid flow and using numerical values taken from the work of Reynolds, Schiller, Jakob, and Erk, has derived the formula:

$$e' = \left[1 - \left(1 - \frac{e}{e_m}\right)\left(\frac{D}{D'}\right)^{c_2}\left(\frac{H}{H'}\right)^{c_2^2}\right] e_m'$$

Staufer's notation being here modified to correspond to that used by the writer. Adopting a value of $\frac{1}{4}$ for the exponent, c_2, this formula then becomes:

$$e' = \left[1 - \left(1 - \frac{e}{e_m}\right)\left(\frac{D}{D'}\right)^{1/4}\left(\frac{H}{H'}\right)^{1/16}\right] e_m'$$

In this formula, e_m denotes the mechanical efficiency of the turbine, that is (expressed in percentage), 100 minus the percentage of loss in mechanical friction in bearings, etc.

It will be noted that the effect of change in diameter as derived by Mr. Staufer is the same as that proposed by the writer. A considerable number of comparative tests investigated by the writer indicate that the exponent of the ratio of heads should be even smaller than the figure of $\frac{1}{16}$ derived by Mr. Staufer. Unless the ratio of heads is very great the change in head is of only minor effect. Since the mechanical losses in most turbines of ordinary

* Cons. Engr., I. P. Morris, Dept., The William Cramp & Sons Ship & Engine Bldg. Co., Philadelphia, Pa.
† Received by the Secretary, November 23, 1925.
‡ *Proceedings*, Am. Soc. C. E., August, 1925, Papers and Discussions, p. 999.
§ *Zeitschrift des Vereines deutscher Ingenieure*, March 28, 1925.
‖ *Die Wasserkraft*, May 1, 1925.
¶ *Proceedings*, Am. Soc. C. E., August, 1925, Papers and Discussions, p. 1009.

sizes are very small, it is believed that these losses need not be separated from the total loss, and it has been found that the simpler form of expression proposed on page 1012* of the writer's paper can be applied satisfactorily to the total efficiency of the turbine with the value of the exponent as there stated, namely, $\frac{1}{4}$.

Mr. Staufer's paper also includes some interesting results of recent tests on a high-speed propeller type turbine.

* *Proceedings*, Am. Soc. C. E., August, 1925. Papers and Discussions.

REPORT OF THE JOINT COMMITTEE ON STANDARD SPECIFICATIONS FOR CONCRETE AND REINFORCED CONCRETE

Discussion*

By L. T. Guy, Assoc. M. Am. Soc. C. E.

L. T. Guy,† Assoc. M. Am. Soc. C. E. (by letter).‡—The specification of the Joint Committee for fine aggregate includes the color test, which is specified in Section 11,§ as follows:

"Sand when tested * * * shall show a color not darker than the standard color, unless it complies with Section 10."‖

Does not the last sentence of this clause render the color test valueless as far as a specification is concerned? It appears that the only ground for the rejection of any material would be its failure to comply with the mortar strength tests.

The color test is, of course, generally used as a preliminary field test, but the writer's experience has led to the conclusion that it is not necessarily a true index of the value of the sand for concreting purposes, unless supplemented by mortar tests. This would appear to be the viewpoint adopted by the Joint Committee, but in this event it is unnecessary to specify the color test.

One sand recently tested for the writer showed a very bad color, while the mortar test showed a strength equal to that of standard sand. The writer believes that organic material may be of such nature as to have an injurious effect on the durability of the concrete, although perhaps not reducing its initial strength below the specified requirements; he, therefore, would hesitate to use a sand showing a bad color test even if it did comply with the mortar strength test. However, the writer has had no actual experience which would substantiate that idea, and, therefore, would much appreciate any information on this question. If this idea is correct it would appear that the aggregate should pass the color test, irrespective of its behavior under the mortar strength test.

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* Discussion continued from December, 1925, *Proceedings*.

† Asst. to Civ. Engr., State Electricity Comm. of Victoria, Melbourne, Victoria, Australia.

‡ Received by the Secretary, November 18, 1925.

§ *Proceedings*, Am. Soc. C. E., October, 1924, Papers and Discussions, p. 1170.

‖ *Loc. cit.*, "Mortar Strength Test".

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

EDWARD ADAM BECK, M. Am. Soc. C. E.*

DIED OCTOBER 18, 1925.

When Edward A. Beck, City Manager of Lynchburg, Va., died in that city on October 18, 1925, the profession to which he had dedicated his life lost one of its ablest and most successful members. Although he was only forty years of age at the time of his death, his accomplishments were already conspicuous in the field of municipal administration, and he had reached the position where he was regarded as a leader in this important branch of modern city government.

Edward Adam Beck was born on January 19, 1885, in Peru, Ind., the son of John Henry and Catherine Magdelan (Silberman) Beck.

After attending the local elementary schools, he entered Purdue University, from which he was graduated in 1913 with the degree of Bachelor of Science. In 1917, he received the degree of Civil Engineer.

From 1906 to 1914, prior to and during his college course, Mr. Beck was engaged on engineering work, ranging from preliminary surveys for local electric lines to the construction of important railroad terminals. During five years of this period he served as Division Engineer of the Wabash System and had supervision of the location and construction of the important terminal at Detroit, Mich., including the negotiation for municipal franchises, a valuable training for the profession which he afterward adopted, and in which he was a real pioneer.

In 1914, he was called to the Borough of Edgeworth, Pa., as Borough Manager. He was the first to hold such a position in Pennsylvania and was able to do so only under a very liberal construction of the then existing laws. The experiment was, however, so successful, that general laws were promptly enacted by the State, permitting the adoption by municipalities of the manager form of government. During 1917, he was also Manager of Sewickley, a borough contiguous to Edgeworth. It is doubtful whether this unique distinction of serving two cities in such capacity at the same time has been enjoyed by any other City Manager.

When Mr. Beck resigned these positions, he was elected, from among a large number of aspirants, to the position of City Manager of Goldsboro, N. C., being the first official in this position for that city. During the period of approximately two years that he served in this capacity, he displayed marked ability as an administrative officer, and gained much valuable experience in the conduct of municipal affairs.

In 1918, he resigned his civil position to enter the Army during the World War. Commissioned first as Captain in the Sanitary Corps, he was later

* Memoir prepared by R. W. B. Hart, Assoc. M. Am. Soc. C. E.

promoted to the rank of Major in the Engineer Reserves of the United States Army and was placed in charge of water supply, sewage disposal, and mosquito control, at Camp Shelby, Miss. He was under orders for overseas service when the Armistice was signed.

After his discharge from military service Mr. Beck again entered the field of municipal management and was elected as City Manager of Auburn, Me., where he also held the position of Chief Engineer of the Water and Sewer District. So marked was his success in this position that the municipal authorities of Lynchburg, Va., after a careful survey of the entire field, of available candidates, elected him as its first City Manager. He entered upon the duties of his office in Lynchburg on September 1, 1920, and continued in this service until his death, which was caused by pneumonia, following an operation.

Mr. Beck brought to the administration of the affairs of Lynchburg an experience that peculiarly fitted him for the tasks there presented, having displayed such unusual ability, untiring energy, and genius for organization and management that he won the entire community to the ardent support of the new plan of government, although a number of its citizens had at first bitterly opposed the change. His record of accomplishment in Lynchburg not only gained recognition from the citizens of that community, but attracted the attention of those interested in city government in all parts of the United States, and he became definitely established as a distinguished leader in this new profession.

At the time of his death Mr. Beck had just completed the preliminary work necessary for the immediate rebuilding of Rivermont Bridge in Lynchburg. The work involved the change from an iron structure to one of reinforced concrete. The idea was his own and resulted in the saving of many thousands of dollars to the city. Because of the unusual height and length of the viaduct it is, perhaps, the most monumental work of this character ever undertaken and has attracted the interest of engineers in all sections of the country. In grateful recognition of the valued services of its first City Manager, and as a tribute that will most appropriately perpetuate his memory, the City Council has formally designated this structure, "Beck Bridge," and on its completion a suitably inscribed tablet will be unveiled with appropriate public ceremonies.

Mr. Beck was noted for the thoroughness of his knowledge of all the details incident to the administration of municipal affairs. He was, therefore, a man of prompt decision and of unusually sound judgment. The City Council soon learned that it could with confidence follow his recommendations in all matters with the assurance that they had been most carefully and intelligently considered by him.

He was a man of unfailing courtesy and consideration, and of absolute fairness. The humblest citizen could always gain access to him and receive an attentive hearing. The most influential person could not secure a privilege or a concession that he did not deem fair and just to all.

Possessing vigorous mentality and dauntless moral courage, he could maintain his conclusions with forceful ability in any forum, for he was a strong debater. He was often called on to address audiences interested in civic affairs

and his utterances were controlled by a lofty conception of the true ideals of citizenship and public service.

Mr. Beck found a most congenial atmosphere in Lynchburg and frequently stated that he hoped to spend his life among its people. Such a purpose was itself proof of the sincere affection in which he was held by them, and it was further attested when the news of his untimely and unexpected death shocked the city, for the whole community was grief-stricken and expressions of personal sorrow were heard from those in every walk of life.

Business was suspended throughout the city during the funeral ceremonies, and when his body was borne to its last resting place in Spring Hill Cemetery, it was laid in a grave that was banked with beautiful floral designs, tributes from nearly every organization in the city and from hundreds of individuals, while throngs of people assembled to share in this expression of a common sorrow for a great loss.

Although young in years, his life had been rich in service and filled with the joy of worth while achievement. On the people of every community in which he had lived, he left the impress of a clean and exemplary life devoted to the highest ideals. The influence of such an example never dies; no man can leave a richer legacy to posterity.

On June 26, 1907, he was married to Hazel Maude Locke, of Peru, Ind., and is survived by her and one son, Edward Adam Beck, Jr., as well as by a sister Mrs. Catherine Dilly, of Cleveland, Ohio, and a brother, Albert J. Beck, of Indianapolis, Ind.

Mr. Beck was a man of rare personal charm and one who delighted in the intimate associations of his fellows. He was a member of the Masonic Fraternity, having been a Thirty-second Degree Mason, a Knight Templar, and a Shriner. He was also an Elk and a Rotarian and was very active in the latter organization.

Mr. Beck was elected an Associate Member of the American Society of Civil Engineers on May 6, 1914, and a Member on November 26, 1918.

ENRIQUE BUDGE, M. Am. Soc. C. E.*

DIED JUNE 2, 1924.

Enrique Budge was born in Valparaiso, Chile, on November 15, 1844, the son of Robert Forbes Budge, a native of Scotland, and Mercedes Prats y Urizar. Robert Forbes Budge was an *ex*-officer of the British Navy, who after the surrender of Napoleon Bonaparte retired from the service. He made two voyages to Chile, the first in 1818 and the second in 1820. He then remained in that country and, after a few years, married Mercedes Prats, the daughter of the Postmaster-General of Chile. Enrique Budge was the seventh son of a family of ten. He began his education at the English school of Messrs. Bloom and Goldfinch in Valparaiso, specializing in mathematics.

Mr. Budge began his technical career in 1863 at the Victoria Foundry and Iron Works of Messrs. Henderson, Flynn and Company, of Valparaiso, where

* Memoir prepared by Edward B. Budge, M. Am. Soc. C. E.

he acquired a practical knowledge of structural engineering that was of great assistance to him in his subsequent work. In 1864, he joined the forces of the Valparaiso and Quillota Railway, then under construction by the renowned English engineer, George Lloyd. In 1865, he was appointed by the Chilean Government as Assistant Engineer on the San Fernando and Curico Railroad and, in 1867, undertook the work of turning the course of the Rio Teno near Curico. This river, due to extraordinary freshets, had inundated the neighboring farms, and Mr. Budge opened a new course for its flow by cutting the railway embankment, thereby completely isolating the railway bridge. This work was most successful.

From 1867 to 1869, he was Resident Engineer of the work at Paso Hondo, on the Valparaiso and Santiago Railroad. This work included difficult alignment of the track, which crossed the Paso Hondo River five times in a distance of 800 m. It also included the construction of heavy embankments, cuts, and tunnels to obtain the most economical layout which called for a single bridge and a tunnel to cross the stream, at a consequently low maintenance cost, which was the end desired. As the old bridges had been built ten years previously of timber with arched spans and were of different sizes, a serious difficulty was encountered because of the enormous expense entailed, including the delay of traffic. The new location, therefore, was accepted as a solution inasmuch as it did not interfere with the traffic. Having completed this work, he was appointed in 1869 as Resident Engineer of the Valparaiso and Santiago Railroad.

From 1870 to 1872, as Resident Engineer, Mr. Budge surveyed and constructed the Los Vegas and San Felipe Railroad, a branch of the main line from Valparaiso to Santiago, Chile, and from 1872 to 1874, he extended that branch line 45 km. to Los Andes, Chile, the starting point of the Chilian Transandine Railway to Argentina through the Cordillera de los Andes, *via* Mendoza. This railroad completed, he was again in charge, as Resident Engineer, of the Valparaiso and Santiago Railroad, including the Los Andes Branch.

In 1875, the Chilian Government awarded by contract the construction of the Curico and Angol Railroad and the Chillan, Concepcion and Talcahuano Railroad, a total length of 425 km. Technical and financial difficulties began with the contractors and the bankers who were to provide the funds for the work, and, consequently, with the Government itself. Mr. Budge, whose sound judgment and knowledge of railway construction had already greatly helped the Government on similar occasions, was proposed as Chief Engineer and Controller of Finances of the enterprise, and was immediately accepted in this capacity by mutual agreement of the three parties concerned. Work on this part of the Chilian Railway System was most difficult and expensive, as the railway extends to the southern part of the Republic and crosses all the rivers of the country, which flow from east to west and are subject to heavy freshets in winter. Moreover, as the large bridges included in the contract were to be temporary on account of the cost involved and the time required for permanent structures, and yet had to be of sufficient strength to withstand the traffic during the winter, the contractors were at a loss as to how to comply with the

Government requirements; but, fortunately, Mr. Budge's perseverance and sound advice overcame all difficulties.

In 1878 and 1879, he was commissioned by the Chilian Government to study the construction and working of railroads in Europe and the United States, but on account of the war with Peru and Bolivia, he was obliged to return to Chile before finishing his commission, having visited only European countries. He was again appointed Resident Engineer of the Valparaiso and Santiago Railroad in 1880, and in 1881 he was appointed Chief Engineer and Director of the new Government mole at Valparaiso, a very important and complicated work which required a foundation of fifty-three caissons, from 20 to 100 ft. long, and 11½ ft. in diameter, sunk by the pneumatic process and filled with mortar. On this foundation iron bridge girders were anchored, and a system of smaller girders supported a concrete pavement laid on steel buckle-plates. In this pavement were embedded the tracks for the heavy hydraulic cranes used for unloading material directly from the ships, and also tracks for hauling loaded and empty cars. A 60-ton hydraulic crane was erected for handling extra heavy loads. This mole has stood as a monument to Mr. Budge's enterprise, for neither storms nor the great earthquake of 1906 did the least damage to the solid foundations of this unique structure. During these years, he also studied and carried out plans for the water-works systems of the Towns of Curico, San Felipe, and Los Andes.

In 1884, he was called upon by the Chilian Government to take charge as Engineer in Chief of the whole plan for Government railways from Valparaiso to Talcahuano and Angol. He, therefore, undertook the work of reconstructing all the important temporary bridges, which, having been built under economical conditions, were actually a great hindrance to railway traffic, particularly as in winter heavy freshets were sure to destroy some of the bridges and embankments. Among many others, he reconstructed the Nuble Bridge, the Laja Bridge, and the Bio-Bio Bridge, each 500 m. long, in 50-m. spans. The abutments and piers were built by the pneumatic process because these rivers carry such a volume of water that no other form of foundation was considered satisfactory.

Mr. Budge continued his connection with the Chilian State Railways until 1894, and, during that time, he served as Director-General of the State Railways in 1885, 1891-92, and 1894. His work on these railways was of the utmost importance, and involved, in addition to the construction of bridges as previously mentioned, new lines, stations, etc., making improvements in accordance with the latest engineering practice.

He had been a member of the Sociedad de Fomento Fabril (an organization of merchants and manufacturers formed to promote Chilean business interests), since 1883, and served as Counselor in 1889; Vice-President in 1898; and President from 1902 to 1908. Due to his important and long service he was nominated in 1909 as Honorary President, which office he held until his death. During his term of office, he organized three technical schools in Santiago, and one in each of the following towns: Talca, Chillan, Concepcion, Valdina, Serena, and Iquique.

In 1901, Mr. Budge undertook the work of preparing an exhibit to be entered by the Chilian Government at the Pan-American Exposition, at Buffalo, N. Y., and was given charge of this task as General Commissioner for Chile. It was due to his unfailing energy and propaganda in favor of this plan that his country was able to hold a most merited and flattering position in that World's Fair.

In 1904, he was again appointed by the Government to take charge of the Chilean National Exposition, which was carried out principally by the Sociedad de Fomento Fabril and was a great triumph for Chilean industries. The thanks of these industries were tendered Mr. Budge who had devoted all his experience to the success of the undertaking.

He acted as Consulting Engineer to Messrs. Clarke Brothers who had concessions from the Chilean and Argentina Governments for building the Transandine Railway to Argentina. When the proposed plans were submitted to him, he at once offered a complete modification of them. In order to use the common steam locomotive to reduce the general gradient of 8%, the proposed plans had included a series of costly loops and tunnels. Mr. Budge at once recommended the higher grade and the use of the Abt rack system. The contractors immediately grasped the idea and accordingly new plans were drawn using a grade of 8%, with rack line, which resulted in great economy in cost and time of construction. This line crosses the Andes at a height of 10 500 ft., through a tunnel 3 144 m. long.

Mr. Budge acted on behalf of the Government as Consulting Engineer to report on the proposal for the construction of the graving dock at Talcahuano; he also reported on the construction of wharves and quay walls for the Port of Iquique and a mole for Pisagua.

The terrible earthquake at Valparaiso in August, 1906, which destroyed this and neighboring cities and resulted in enormous losses through death and fires caused by falling houses, introduced a most serious problem as regards the reconstruction of the city. Mr. Budge was called on by the Government to undertake this work which required an experienced engineer. New plans had to be laid out, new grading from hills to the sea-front, new outlets for the streams, new streets—in short, a new city had to be designed with modern causeways and footpaths, with an expenditure of £3 000 000 gold.

Having completed this work of reconstruction, Mr. Budge was nominated in 1910 as Inspector General of Materials to be furnished to the Chilean Government by European and North American firms. He devised a sound method of financial accounting for materials provided to the railways and Government contractors, including those used in the construction of quay walls and breakwaters at Valparaiso, by the firm of S. Pearson and Son, Limited. He held that office until 1920, during which time he visited the principal manufacturing establishments of Europe and the United States.

On his return to Chile, Mr. Budge was appointed Director of various industrial corporations, which positions he held until his accidental death. In his endeavor to pass through a crowd in one of the banks in Santiago, he fell into a deep trench recently opened for the layout of electric cables, striking his

chin. This blow caused concussion of the brain which gradually developed into paralysis and subsequent death.

Mr. Budge was very active, never tiring of any work that he undertook. He was the first President of the Portland Cement Factory, "El Melon", the only factory of its kind in Chile, and placed it on a sound financial basis. It can be truly said of him, that up to 1910 no engineering plan was carried out by the Government of Chile or by Chilean contractors without his endorsement, so that his name is connected with almost all the public works constructed in Chile during the last fifty years.

Mr. Budge is survived by his widow, Mrs. Carolina Zañartu de Budge, and three children, Olga, wife of Augustin Edwards, Chilean Minister to Great Britain, Carlos, and Enrique.

He was elected a Member of the Institution of Civil Engineers of Great Britain and also a Member of the Institution of Mechanical Engineers in 1879. He was a faithful member of the Roman Catholic Church.

Mr. Budge was elected a Member of the American Society of Civil Engineers on February 1, 1882.

STEPHENSON WATERS FOX, M. Am. Soc. C. E.*

DIED MAY 8, 1925.

The unfortunate sinking of the Steamer *Norman* in the Mississippi River below Memphis, Tenn., on May 8, 1925, took a large toll from the membership of the Society. Of this number was Stephenson Waters Fox, of Paducah, Ky.

"S. Waters" Fox, as he preferred to be known, was born in Natchez, Miss., on August 3, 1856. He was graduated from the Rensselaer Polytechnic Institute, Troy, N. Y., in 1876 as a Civil Engineer. Previous to and following his graduation, Mr. Fox, like many young engineers of that day, obtained his early experience on railroad surveys, these engagements being on several railroads in Southern Indiana, Kentucky, and Tennessee.

In 1876, he was for a short period with the Snead and Company Iron Works, of Louisville, Ky., as Computer. In 1878, he became an Assistant to the late Brig.-Gen. Charles R. Suter, U. S. A. (*Retired*), M. Am. Soc. C. E., then Major, Corps of Engineers, U. S. A., and was engaged on river improvement work on the Mississippi River. He remained as an Assistant Civil Engineer with the U. S. Corps of Engineers, until 1903. During this period he had a large degree of responsibility for the design and construction of many important river improvements, principally of revetments for the protection of caving banks and levees, and the control of the Mississippi and Missouri Rivers and their tributaries, including works at Choctaw Bend, Mississippi River, and at Glasgow, Cedar City, St. Charles, St. Joseph, Kansas City, Mo., and other points on the Missouri River.

* Memoir prepared by Charles F. Loweth, Past-President, Am. Soc. C. E.

In 1903, Mr. Fox resigned from the Government service and opened an office at Kansas City, Mo., as a Consulting Engineer, specializing in river improvements, the construction of levees, and land reclamation. In this capacity he served, with much credit, several railways, municipalities, and other corporations, on difficult projects of considerable magnitude and also designed several large suction dredges. During this period, he was for several years Chief Engineer for the Union Terminal Railway Company, of St. Joseph, Mo. Later, he moved to St. Louis, Mo., where his engineering activities were of similar character.

In 1917, when the United States entered the World War, Mr. Fox was commissioned a Major in the Engineer Reserve Corps, and served with credit in the Division of Inland Waterways of the United States Railroad Administration as Engineer of Contracts and Construction. After the close of the war Major Fox became associated with Cox and Stevens, Marine Engineers, of New York, N. Y., and while with this firm designed a new type of boat for use on the Erie Canal.

In 1922, he became Consulting Engineer for the Joyce-Watkins Company, large producers of railroad ties, telegraph poles, etc. This engagement necessitated his removal to Paducah, Ky. In addition to the questions of timber treating and of transporting ties and poles by river, he had presented to him many new and novel problems, such as the design of special machines and floating equipment for handling a special product. To these he brought to bear those qualities of patient, thorough, and open-minded investigation which were characteristic of him and which had contributed largely to his success and had won for him the hearty commendation of many employers. His zeal and interest in new and novel, yet always practical, problems kept him much younger than his years.

Major Fox was considered an authority on levees, bank protection, and other kindred problems relating to the control of the Missouri River, to which he had given much careful and enthusiastic study. In 1905, he presented to the Society a noteworthy paper entitled "Technical Methods of River Improvement as Developed on the Lower Missouri River, by the General Government, from 1876 to 1903."*

Outside his professional work, Major Fox always found time and interest for many worth-while activities and lent his assistance to many enterprises that promised good to the community. Although a resident of Paducah for only three years, he was, at the time of his death, President of the local Lions Club; he had been for some time a member of the City Planning Commission and had been appointed its Chairman within a month of his unfortunate death. For many years he was an active member of the Presbyterian Church.

Major Fox was a man of culture and appreciation, fair-minded, broad, and kindly. He had a genial and happy disposition and made and retained many friends. If one may be judged by his aspirations and ideals, as well as by his accomplishments, the sentiments in the following verses, a typewritten copy of which was found on his remains, reflects in some degree the measure of the man:

* *Transactions*, Am. Soc. C. E., Vol. LIV (1905), p. 280.

"Carve your name high o'er the shifting sands,
Where the steadfast rocks defy decay.
All you can hold in your cold, dead hand
Is what you have given away.

"Count your wide conquest o'er sea and land,
Heap up your gold, hoard as you may.
All you can hold in your cold, dead hand
Is what you have given away.

"Build your pyramids, skyward, let them rise,
Stand gazed at by millions, cultured they say
All you can hold in your cold, dead hand,
Is what you have given away.

"Silver and gold and jewels, all so grand,
King of the solon, a mart, a day.
Yet all you can hold in your cold, dead hand
Is what you have given away."

The great river on whose banks Major Fox was born and spent his boyhood days, and near which most of his life was spent, finally claimed him as its own; but the works in the design and construction of which he had so large a share, built to control the river, are yet unconquered by it.

He is survived by his wife, a son, Burton C. Fox, of Portland, Ore., and three daughters, Mrs. Jett Blackburn, of Portland, Ore., Mrs. Cleveland X. Hening, of San Francisco, Calif., and Mrs. Harvey J. Schmelzer, of Kansas City, Mo.

Major Fox was elected a Junior of the American Society of Civil Engineers on July 7, 1880, and a Member on October 6, 1886.

CLARENCE LORING HUSSEY, M. Am. Soc. C. E.*

DIED DECEMBER 5, 1925.

Clarence Loring Hussey was born in Milford, Mass., on November 16, 1884. He was the son of the late Albertus and Effie A. (Holmes) Hussey, both of whom were born in Milford, where Mrs. Hussey lived until a few years ago.

Mr. Hussey was graduated from the Massachusetts Institute of Technology in June, 1908. He immediately entered the employ of the Providence Engineering Works as Foreman in charge of automobile experiments and testing, and remained with that firm until March, 1909, when he became a member of the Engineering Staff of F. P. Sheldon and Sons, of Providence, R. I., Mill Construction Engineers. From April, 1910, to July, 1912, Mr. Hussey was employed by the Rhode Island Electric Car Company on designs and experimental work relative to street railway power plants. While with this company he designed and supervised the construction of extensive car barn facilities and several bridges.

* Memoir prepared by G. H. Henderson, M. Am. Soc. C. E.

From July, 1912, until his death Mr. Hussey served as Bridge Engineer for the Rhode Island State Board of Public Roads and as such supervised the design and the construction of all bridges on the State Highway System. In addition, he conducted the investigation for and designed the proposed $3 000 000 Washington Bridge over the Seekonk River, which is to connect the City of Providence with the Town of East Providence. Previous to his death he was directing the examination and preliminary design for a proposed toll bridge across Mount Hope Bay to connect the Island of Rhode Island, on which is located the City of Newport, with the mainland at Bristol. The construction of this bridge is being promoted in order to provide a direct route between the Cities of Providence and Newport.

Among the many original ideas which Mr. Hussey introduced in the construction of concrete arch bridges in Rhode Island was the design which allowed approximately a 50% reduction in the volume of abutments and arch rings for a given width of road. This design called for the carrying of a sidewalk and railing on brackets anchored to the arch ring. On the back of these brackets lay inclined spandrel walls which materially reduced the dead load due to the spandrel fill. This form of construction made necessary light sidewalk railings in the design of which Mr. Hussey again exhibited striking originality.

He was particular in the selection of mineral aggregates to insure high strength concrete. His special mixtures for contraction joints permitted a larger range of use of these troublesome features of construction, and whenever it became necessary to combine concrete with other materials his designs were unique and effective.

By nature Mr. Hussey was of a quiet, studious disposition. This manifested itself in his investigations of concrete mixtures as applied to the State bridge work and to the construction of his own home which was of concrete. He originated most of the standards now in vogue in his Department; these included an elaborate file of records, photographs, and cost data. In fact, the whole conduct of the State bridge work became highly systematized under his direction and this in large measure accounted for its success.

He was also a keen student of structural design. His ideas, although marked by striking originality, had the saving virtue of reasonableness. In numerous instances he put them into practice to the great economy of the State bridge work. In his conduct of his Department, he was noted for his rigid, almost inflexible, principles applied alike to his colleagues, to contractors, and to himself.

In spite of a reserved outward manner, within he was most friendly and companionable. This became very evident to those who knew him in social or professional organization work. For several years he was Secretary of the Technology Club of Rhode Island (graduates of the Massachusetts Institute of Technology). He was also active in the Providence Engineering Society. His attitude in these services was always one of helpfulness and his efforts were characterized by an innate thoroughness and conscientiousness. Few demands were made on his help without a willing response and no duties did he undertake without their full accomplishment.

As a mark of respect to the memory of its late Bridge Engineer, the Rhode Island State Board of Public Roads, at its meeting on December 9, 1925, adopted unanimously the following resolution:

"*Whereas,* it hath pleased Almighty God in his divine wisdom to call from his earthly labors Clarence L. Hussey, our friend and co-worker; therefore, be it

"*Resolved* that the State Board of Public Roads record its sense of severe loss in the decease of a highly efficient and conscientious member of its staff. Throughout his thirteen years of association with the State Board of Public Roads he performed faithfully and well every task to him assigned. He rendered a public service which may justly be pointed to with pride and held up as an inspiration to others called into public service; and be it further

"*Resolved* that this Board does hereby express to the family of the late Clarence L. Hussey its most sincere sympathy in their bereavement, which loss is shared by the members of this Board in a very personal sense; and be it further

"*Resolved* that this resolution be spread upon the minutes of this meeting and that the Clerk be and he hereby is directed to transmit a copy hereof to the family of our late respected associate and Chief Bridge Engineer."

For several years previous to his death Mr. Hussey, who was unmarried, resided with his parents in Providence. Besides his mother he is survived by a sister, Mrs. John R. Frazier, of Providence.

He was a member of the Masonic fraternity, in addition to local technical societies, and served on various bridge structure committees with the American Association of State Highway Officials and the National Research Council.

Mr. Hussey was elected a Member of the American Society of Civil Engineers on April 7, 1924.

VIRGIL GEORGE MARANI, M. Am. Soc. C. E.*

DIED NOVEMBER 2, 1925.

Virgil George Marani was born on July 4, 1868, at Regio, in the Province of Emilia, Italy. He was left an orphan when very young and, with his two brothers, was removed to Edinburgh, Scotland, where he received a common school education and was taught to forget his native tongue. At the age of eleven, young Marani ran away from school, but was brought back and by way of discipline and punishment was sent to a naval schoolship known as H.M.S. *Conway,* located at Liverpool, England. He served on this ship for two years and then was given an honorable discharge because he was considered to be of insufficient physical strength to complete the naval course. Determined to build up his physical strength and to see the world, the thirteen-year old boy shipped on a full rigged sailing ship of the British Merchant Marine which, at that time, was one of the largest in the world, and began a career as a sailor which lasted until he was twenty-one years of age. In

* Memoir compiled from information furnished by H. H. Macdonald, Secretary, The Gypsum Industries, Chicago, Ill.

that time, he had served on sailing vessels and steamships, had circumnavigated the globe twice, and had sailed many times around Cape Horn and the Cape of Good Hope.

On coming of age, in 1889, Mr. Marani inherited his share of his mother's estate and had the good judgment and the ambition to use this money for his education through high school and university. Within four years he had completed both high school and university work, having been graduated from the University of Toronto in 1893, with the diploma of Civil Engineer. In June, 1924, the degree of Civil Engineer was conferred on him by his Alma Mater.

During his vacations, from 1889 to 1893, Mr. Marani held the position of Chief Inspector of Sewerage Works at Brantford, Ont., Canada, for Willis Chipman, M. Am. Soc. C. E., Sanitary Engineer of Toronto, Ont., Canada, and immediately following his graduation he was engaged as Transitman on survey work for Mr. W. J. White, at Buffalo, N. Y., and, also, as Chief of Field Party and General Inspector for Mr. Edward F. Ball.

Within a year after his graduation from the University of Toronto, however, Mr. Marani went to Cleveland, Ohio, where he was employed in the City Engineer's Office, successively, as Assistant City Draftsman, Chief Draftsman, and Inspector and Engineer of Sanitation. In 1895, he became Assistant Engineer with the Cleveland Gas Light and Coke Company, and in 1900 was promoted to be Engineer of Construction, holding this position for six years. In this capacity he detailed and planned all buildings, machinery, foundations, etc., for a plant covering more than eleven acres.

From 1907 to 1909, he was Engineer Superintendent of the Cuyahoga County Court House, at Cleveland, Ohio, a monumental structure costing $5 000 000. Following the completion of this building in 1910, he was appointed Building Commissioner of the City of Cleveland, which office he held for one year. During this time he revised the City Building Code, established a fee system, etc.

In 1912, Mr. Marani engaged in private practice and designed and superintended the construction of buildings of all types. He also served as Consulting Engineer for the National Fireproofing Company, of Pittsburgh, Pa., and until 1918 as Consulting Engineer for the United States Gypsum Company, leaving this Company to serve during the World War as Representative of the War Service Committee on Gypsum. From 1918 until his sudden death at his home in Evanston, Ill., Mr. Marani was engaged as Chief Engineer of The Gypsum Industries.

Mr. Marani was among the foremost authorities on gypsum in the United States. He was a prolific writer on this subject as well as on construction materials, building codes, fire-protective requirements, etc. Endowed with a genial and friendly personality, he was regarded with affection and admiration by all with whom he came in contact.

Mr. Marani was a member of the American Society for Testing Materials, the American Association of Engineers, Western Society of Engineers, Cleveland Engineering Society, and the National Fire Protection Association. He

was also a member of the Masonic Fraternity, and of the Lake Shore Athletic Club and the Old Colony Club.

Mr. Marani was elected a Member of the American Society of Civil Engineers on February 4, 1914.

MANSFIELD MERRIMAN, M. Am. Soc. C. E.*

DIED JUNE 7, 1925.

Mansfield Merriman, the son of Mansfield and Lucy Merriman, was born at Southington, Conn., on March 27, 1848. His father was a descendant of Captain Nathaniel Merriman, who came from England to Boston, Mass., in 1632, settled in New Haven, Conn., about 1640 and was, in 1670, one of the founders of Wallingford, Conn. His mother was a descendant of John Hall, who came from England to Boston in 1633, settled in New Haven in 1639, and who also was one of the founders of Wallingford.

Mansfield Merriman received his early education at the South Center District School and Lewis Academy, both situated at Southington. In 1866, he left school and went to work on his father's farm. He was employed later in a factory and printing office, taught a country school, studied geometry, surveying, and mathematics in his spare time, and on April 5, 1867, was appointed a County Surveyor. Playfair's Euclid was his first technical book, but he early formed the habit of regularly buying books. During 1866 on a total income of $172 he acquired thirty books at a cost of $27.50. These books included such titles as "Elements of Philosophy," "Letters on Astronomy," "First Lessons in Greek," "Latin Prose Composition," "Lady of the Lake," Blair's "Rhetoric," and Flint's "Surveying." While engineering and mathematics claimed most of his effort, yet his leisure time was devoted in a surprising degree to literature, to history, and to poetry. Along these lines as well as in botany, geology, physics, chemistry, and genealogical research, he found pleasure and recreation.

In spite of his father's stern opposition, but with the sympathetic aid of his mother, he went to New Haven and passed the entrance examinations at Sheffield Scientific School in 1868. He received second prize for excellence in studies during his Freshman year and a prize for excellence in botany in his Senior year, was President of his Class in his Junior and Senior years, and was elected a member of Book and Snake.

After graduating with the degree of Bachelor of Philosophy, in 1871, Mr. Merriman studied as a Graduate Student and received a second degree, Civil Engineer, in 1872. He became an Assistant in the United States Engineer Corps, in which position he remained for a year. Later, he spent six months studying in Germany, at Hanover, Dresden, and Berlin. Returning to New Haven, he was engaged on bridge construction for several months and became Assistant in Civil Engineering at Yale University in 1874, which position he held until 1877. During these years he resumed his studies at the

* Memoir prepared by the following Committee: Horace Andrews, *Chairman*, Walter R. Okeson and A. B. Hill, Members, Am. Soc. C. E.

Yale Graduate School, receiving the degree of Doctor of Philosophy, in 1876. In 1877 and 1878 he was Instructor in Civil Engineering and Astronomy at Yale.

In the fall of 1878, Mansfield Merriman went to Lehigh University, Bethlehem, Pa., as Professor of Civil Engineering. Engineering education was then in its infancy, with few textbooks and with little known as to the proper technique of teaching. With his advent at Lehigh began Professor Merriman's great contribution to engineering education, first, in the form of teaching methods that rendered a complicated subject so simple that boys in his lecture-room were astonished to find his conclusions already formulated in their minds before he stated them; and, second, by the creation of a series of textbooks quite unequalled at the time of their publication, and many of which, after thirty or forty years, are still in daily use in colleges. This latter fact was made possible by the careful and constant revision given to these textbooks, many of which ran through numerous editions.

Perhaps no other man contributed so much to the success of Lehigh University as an engineering school, or had so great an influence in the establishment of her ideals and practices. His own rapidly advancing reputation as a teacher and engineer did much to attract favorable attention to this young school, and the high standards set in his Department induced similar standards in all the other Engineering Departments.

At the Annual Convention of the Society in the City of Mexico, in 1907, Professor Merriman was present, and his reception must have proved very gratifying to him, as it was most enthusiastic, especially at the assemblages in the Mineria, where many of his former pupils had again the privilege of meeting him. Many of the engineering works shown to the members present at that Convention were carried out by the Spanish speaking students who had formerly been under Professor Merriman at Lehigh University.

Lorenzo Perez Castro, M. Am. Soc. C. E., in *The Revista Mexicana de Ingenieria y Arquitectura* for August 15, 1925, wrote:

"During half a century, Merriman, as a great educational force, unceasingly expounded the principles of directness and simplicity in teaching and instruction. * * * Through natural inclination he loved study from the years of his youth. * * * By his own efforts he became an engineer, a mathematician and a man of science. * * * He had recourse to all the fountains within his reach in the effort to satisfy the divine desire. As steps in the ladder he acquired Latin and Greek as well as French and German, all of which he could read fluently. * * * The remembrance of Merriman and his pure constructive work, so beneficial to his fellow men, will be permanent and lasting. It will live in his pupils and students as in all those who, through generations to come, are to profit by his instruction and writings; and when, in the course of time, his books of applied studies become antiquated through the natural evolution of human knowledge, the master will remain embodied in his subjective life, or life of remembrance, with that worthy legion of men who live for humanity."

What Mansfield Merriman did for Lehigh University in a large way, he did for many other engineering schools indirectly through his textbooks, and the whole Engineering Profession owes a great debt to him.

In 1903, while still at Lehigh, he was a member of a Special Commission appointed by Mayor Low, of New York, N. Y., to report on plans for the Manhattan Bridge across the East River. Professor Merriman was also for four years, 1881-85, Acting Assistant in the United States Coast and Geodetic Survey, in charge of summer triangulation work in Pennsylvania.

In 1906, the University of Pennsylvania conferred on Professor Merriman the degree of Doctor of Science, and, in 1913, the degree of Doctor of Laws was conferred on him by Lehigh University.

In 1907, he resigned his position as head of the Civil Engineering Department at Lehigh University and engaged in consulting practice in New York, specializing in hydraulic problems and bridge design and construction.

Professor Merriman's work as an author and editor began in 1876, his thesis for the degree of Ph.D. at Yale, "Elements of Least Squares", having been published in 1877. He served as Editor-in-Chief of the "American Civil Engineers' Pocket Book," beginning in 1908 (first issued in 1911; Fourth Edition, 1920); Associate Editor of Johnson's (now Appleton's) Universal Encyclopedia, 1892-98, having written and edited the articles on Civil Engineering; Joint Editor with Mr. R. S. Woodward of "Mathematical Monographs" (twenty-one volumes), published from 1896 to 1921; author of "The Theory and Calculation of Continuous Bridges" (1876); "Elements of the Methods of Least Squares" (London, 1877); "The Figure of the Earth" (1881); "Method of Least Squares" (1884; Eighth Edition, 1911), translated into Spanish; "Mechanics of Materials" (1885; Eleventh Edition, 1914); "Treatise on Hydraulics" (1889; Tenth Edition, 1916); "Strength of Materials" (1897; Sixth Edition, 1912); "Elements of Sanitary Engineering" (1898; Fourth Edition, 1918), translated into Japanese; "Precise Surveying and Geodesy" (1899; Second Edition, 1912); "Elements of Mechanics" (1905); "Elements of Hydraulics" (1912); "Mathematical Tables" (1915); "Recreations in Mathematics" (1917), published under the pseudonym of "H. E. Licks"; joint author with John P. Brooks, Assoc. M. Am. Soc. C. E., of "Handbook for Surveyors" (1895; Fourth Edition, 1912); and with Henry S. Jacoby, M. Am. Soc. C. E., of "A Textbook on Roofs and Bridges", in four volumes (1888-98; Fourth Edition, 1917). He also contributed on many occasions to the papers and discussions of the Society and to various technical magazines. *Engineering News-Record*, in its number for April 17, 1924, celebrating the completion of fifty years in its engineering work, contained (page 649), a specially prepared paper from Professor Merriman. The total number of his books printed to 1925 was 340 000.

As a member of various scientific and mathematical organizations, Professor Merriman contributed to their proceedings and also prepared numerous articles for the engineering and technical press. He was one of the founders of the Society for the Promotion of Engineering Education and its President in 1896; one of the original organizers, in 1898, of the American Section of the International Association for Testing Materials, holding the office of Chairman of the organization until 1900; he was, therefore, one of the charter members of the American Society for Testing Materials at the time of its organization in 1902, and was elected President in 1915. He was a Fellow of the

American Association for the Advancement of Science and Vice-President of the Engineering Section in 1894; member of the American Philosophical Society; American Institute of Mining and Metallurgical Engineers; American Mathematical Society; New York Academy of Sciences; Mathematical Association of America; Yale Engineering Association; Wallingford Historical Society; Engineers' Club, New York, N. Y.; re-elected President of the Class of 1871, Sheffield Scientific School, in 1921; member of H. Stanley Goodwin Lodge, F. and A. M., Bethlehem, Pa., and former District Deputy Grand Master.

Professor Merriman's own evaluation of the science of teaching and of his philosophy of life was expressed in an address delivered in 1911 at Southington, Conn., before the Alumni Association of Lewis Academy, when he said:

"Benjamin Franklin published about 1780 an essay on the 'Usefulness of the Mathematics'. This is one of those simple and wonderful articles which that great man prepared for the express purpose of influencing the young men of America, and it explains how mathematics lies at the foundation of surveying, engineering, and astronomy. In it occurs a reference to the 117th proposition of the 13th book of Euclid's Geometry, and Plato is quoted as saying that a man who could not understand this proposition ought not to be ranked among rational creatures. This essay first came into my hands in 1864, and its perusal greatly impressed me, especially the reference to Euclid, and accordingly I sent to New Haven by my aunt, who went there twice a year to buy supplies for her millinery trade, to purchase a copy of Playfair's Euclid, which was then used as a textbook at Yale College.*

"During an experience of more than thirty years, I have had good opportunities for estimating the influence which teachers actually exert. Time only allows me here to say that such influence seems to be slight outside of the class exercises. Teachers who go out upon the bleachers and swing their hats usually lose the respect of students in the class room. Teachers who select students as favorites under the idea of influencing them, put themselves in a most awkward position, which injures all parties concerned. The influence of teachers upon pupils seems to lie almost entirely in training them to proper habits of order and obedience, and in securing from them satisfactory attention to the lessons assigned. Special interest in particular lines of study, high ambitions, and resolves toward forming better habits of life appear to come to a boy not from teachers but from parents, friends, fellow students, and books. On the whole then, it seems to me wise for a teacher to confine himself zealously to the work of instruction, being always strictly impartial in attempting to influence his pupils, if he desires to attain the highest degree of success.

"A good definition of success lies in its contrast with another word. Proceed means to go ahead, but succeed means to go ahead and to get there. With all young men and young women who have formed a purpose, there exists an ambition to succeed. To this end hard work is necessary, and also the results of this work must be a little better than those of the average men and women who are engaged in the same kind of labor. But even these cannot bring success unless the worker rejoices in his work and sees that it brings each year better and better results. True success is not the attainment of money or fame, but rather the joy and satisfaction of the work itself and the consciousness that its results are regarded as valuable by those competent to judge.

* This incident played a direct part in formulating Professor Merriman's determination to go out into that great world which lay beyond the horizon of his ancestral home.

"And the message that I would fain try to bring is this: Have joy in your labor, and then your work shall be well done; perhaps also some of the results of that work shall exert inspiring influences in directions of which you yourself do not dream."

Professor Merriman was married on June 5, 1875, to Wanda Kubale, of Posen, Germany. He is survived by five children, Thaddeus, Lucile, Alice Pauline, Richard Mansfield, and Norman Nathaniel, issue of this marriage; two of the sons, Thaddeus and Richard Mansfield, are members of the Society. Mrs. Merriman died on March 10, 1889, and Professor Merriman was married on May 24, 1891, to Anna Rosina Godshalk, of Bethlehem, Pa., who died on January 8, 1907. He was married for the third time on June 6, 1910, in New York City, to Bazena Treat Downes. There were no children by the second or third marriage.

Professor Merriman was elected a Junior of the American Society of Civil Engineers on May 12, 1875, and a Member on September 3, 1884.

AMBROSE VINCENT POWELL, M. Am. Soc. C. E.*

DIED DECEMBER 5, 1924.

Ambrose Vincent Powell, the only child of Col. George W. and Mary (Dunbar) Powell, was born at Laurens, N. Y., on May 4, 1847. He attended the Cooperstown, N. Y., Seminary and, later, studied at Rensselaer Polytechnic Institute at Troy, N. Y., from which he was graduated in 1868 with the degree of Civil Engineer.

In 1869 and 1870, Mr. Powell was engaged in general engineering work, and, in 1871, became Assistant Engineer on the Oil Creek and Alleghany River Railroad. From 1872 to 1875, he held the position of Engineer of the Michigan and Ohio Railroad and in 1876 and 1877, was engaged in general engineering practice at Battle Creek, Mich.

In 1878, he became Engineer for the Receiver of the Chicago and Northwestern Grand Trunk Railroad Company and, in 1879, was Principal Assistant Engineer of the Chicago and Grand Trunk Railroad. In 1880 and 1881, he was Principal Assistant Engineer of the Chicago and Western Indiana Railroad and in 1882 and 1883, Engineer of the South Chicago Dock Company.

In 1884 and 1885, he was Chief Engineer and Superintendent of the Calumet and Chicago Canal and Dock Company, and from 1886 to 1888, he served as Engineer and Commissioner of Public Works of Hyde Park, then a suburb of Chicago, Ill.

From 1889 until his death, Mr. Powell was engaged in private practice as a Consulting Civil Engineer in Chicago, making a specialty of dock and harbor work. He built the lake front harbor improvements at Ashtabula, Ohio, as well as harbor improvements on the Calumet River for the South Chicago Dock Company and for the Calumet and Chicago Canal and Dock Company. He constructed a yard and dry dock for the Chicago Shipbuilding Company,

* Memoir compiled from information on file at the Headquarters of the Society.

a dry dock for the Superior Shipbuilding Company, and coal-handling plants for E. L. Hedstrom and Company, George Lill Coal Company, O. S. Richardson and Company, and others, at Chicago and elsewhere. He designed and constructed dry docks for several large shipbuilding companies on the Great Lakes, at Buffalo, N. Y., Toledo, Ohio, and at other lake cities. He was the first to use a concrete portal for a dry dock.

Mr. Powell was a successful engineer. His keen observation and logical thinking resulted in sound engineering judgment. His citizenship was always thoughtful and constructive; for many years he was an active member and officer of the Hyde Park Improvement Association, a voluntary organization which was concerned with matters relating to the welfare of that section of Chicago.

He was a member of the Western Society of Engineers for forty-three years having served as Treasurer in 1887. In 1898, Mr. Powell became Vice-President, and, in 1900, was elected President. He was a member of the Calumet Country Club, the Chicago Engineers Club, and the Union League Club of Chicago. He is survived by his wife, Hannah Belle Clark Powell.

Mr. Powell was elected a Member of the American Society of Civil Engineers on February 6, 1901.

ROBERT HUGH McNEILLY, Assoc. M. Am. Soc. C. E.*

DIED MAY 8, 1925.

Robert Hugh McNeilly was born on September 30, 1879, near Hustonville, Lincoln County, Ky. His father was Dr. James H. McNeilly, a Presbyterian minister and a former Captain and Chaplain in the Confederate Army. His mother was Mary Weatherford McNeilly.

Mr. McNeilly received his engineering education at Vanderbilt University, Nashville, Tenn., from which he was graduated with the degree of Bachelor in Engineering, in 1902 (conferred in 1908). He received the degree of Civil Engineer from the University in 1915.

He was employed as a Draftsman in the Ordnance Department, United States Army, until 1904. From 1904 to 1905 he was Constructing Engineer for several water supply and hydro-electric plants in Tennessee and Mississippi, and from 1905 to 1906, he was Assistant Professor of Civil Engineering at the University of Mississippi, after which he was again employed by the U. S. Army in the capacity of Engineer and Superintendent of Construction on the Columbus, Ohio, Barracks from 1906 to 1907. During 1907, he held the position of Assistant Engineer for the Muscle Shoals Hydro-Electric Power Company.

From 1907 until his death, Professor McNeilly was a member of the Engineering Faculty at Vanderbilt University and during that time he also conducted a private engineering practice.

* Memoir prepared by Hunter McDonald, Past-President, Am. Soc. C. E., and L. R. Currey, Jr., Assoc. M. Am. Soc. C. E.

It was in his capacity as a teacher that Professor McNeilly accomplished most for his students and friends. He had a singularly happy and cheerful disposition, was efficient, earnest, and just, and always ready to help in any task or problem where he might be needed. He was the moving spirit in the organization of the Student Chapter of the Society at Vanderbilt University, which has been named the Robert H. McNeilly Chapter in honor of him.

Professor McNeilly was drowned in the *Norman* tragedy in the Mississippi River, near Memphis, Tenn., on May 8, 1925. He was married to Marian Brown Stockell on June 8, 1904, and is survived by Mrs. McNeilly and a son, Stockell McNeilly.

In his address at the memorial services to the Vanderbilt University Alumni who perished in this disaster, Chancellor Kirkland said with respect to Professor McNeilly: "He helped lift the burdens of others. He did not put his burdens on you."

Professor McNeilly was elected an Associate Member of the American Society of Civil Engineers on November 26, 1923. He was also a member of the Engineering Association of Nashville.

ALBERT FRANKLIN SICKMAN, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 17, 1925.

Albert Franklin Sickman, son of John and Martha Pomerene Sickman, was born at Snowden, Pa., on December 10, 1856. He received his technical education at the University of Pittsburgh, Pittsburgh, Pa., from which he was graduated in 1878.

In July, 1878, Mr. Sickman went to Holyoke, Mass., and entered the service of the Holyoke Water Power Company, remaining in the continuous employ of that Company until his death—a period of forty-seven years.

He went to Holyoke at the invitation of his brother, Mr. James M. Sickman, then the chief engineering member of the office force of the Holyoke Water Power Company. This organization, however, at that time, did little engineering work, being mainly engaged in giving lines and grades for new streets and in general surveying. Soon afterward, in 1879, the Company undertook the organization of a systematic control and lawful allotment to each mill owner of the quantity of water he should draw from the Company's canals—the common receptacle from which they all drew the water that was then converted into power by each mill owner.

For lack of such control and allotment, water was being largely overdrawn, wasted, and was even at work in a manner to cause the destruction of some of the canals. This called for the services of an hydraulic engineer, and, as a pupil of the late James B. Francis, Past-President, Am. Soc. C. E., of Lowell, Mass., Clemens Herschel, Past-President, Am. Soc. C. E., was chosen for this work.

* Memoir prepared by Clemens Herschel, Past-President, Am. Soc. C. E., and Robert E. Barrett, M. Am. Soc. C. E.

Mr. Herschel went to Holyoke in 1879, and "Frank" Sickman, as he was generally called, was designated to be Mr. Herschel's first Assistant. At that time, at twenty-three years of age, Mr. Sickman was the same quiet, conscientious worker that he remained to the day of his death. He had been firmly grounded in the elements of his profession, by faithful study at the University of Pittsburgh, and Mr. Herschel takes pleasure in recalling his ready response, as a most apt pupil, to every demand made on him.

When Mr. Herschel resigned his position as Hydraulic Engineer of the Holyoke Water Power Company, in 1889, Mr. Sickman became his successor. At that time the system of charging for excess drafts of water by the mill owners, and thus keeping control of the draft by each one of them, had become firmly established, but naturally required expert continuance of the work. The far-famed Holyoke Testing Flume had been established in 1881, and called for skilled work in meeting the needs of American wheel builders. No one thing has contributed so much, probably, to the present eminent and useful state of the art of hydraulic turbine building in the United States, as the work done in this flume since its foundation, and, to a large extent, by Mr. Sickman after 1889.

In October, 1913, he was elected Assistant Treasurer of the Company and, in November, 1919, was also made Clerk, both of which positions he held at the time of his death.

He was a master of hydraulic problems and familiar with every detail of the history and developments of the work at Holyoke, and in his death, the Company that he had served so long and faithfully suffered an irreparable loss.

Mr. Sickman had varied interests outside his profession. He was a keen student of Nature, an authority on the New England trees, and knew every wild flower of the section, and where and when to find it.

He was always courteous and affable and his genial disposition endeared him to his associates; he was ever fair and considerate in his relations with those under him.

Mr. Sickman was twice married. His first wife died many years ago, leaving four children, all of whom survive him. In 1909, he married Margaret Griffin, of Holyoke, who also survives him.

He was a member of the New England Water Works Association and of the Engineering Society of Western Massachusetts.

Mr. Sickman was elected an Associate Member of the American Society of Civil Engineers on March 4, 1903.